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CONSTRUCTING SLABS FOR TESTS OF JOINTS

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STRUCTURAL EFFICIENCY OF TRANSVERSE WEAKENED-PLANE JOINTS

BY THE DIVISION OF PHYSICAL RESEARCH, PUBLIC ROADS ADMINISTRATION

Reported by **EARL C. SUTHERLAND**, Senior Highway Engineer and **HARRY D. CASHELL**, Associate Highway Engineer

FIGURE 1.—THE FULL-SIZE PAVEMENT SLABS IN WHICH THE WEAKENED-PLANE JOINTS WERE INCORPORATED.

FOR SOME YEARS it has been recognized that the formation of transverse cracks in concrete pavements can be eliminated only by the introduction of closely spaced transverse joints. Consequently, there has been a gradual trend toward decreased slab lengths and in the past 15 years, toward the use of transverse weakened-plane or dummy joints as a means of dividing the pavement into slabs of short lengths.

Early in 1940 the Public Roads Administration sponsored a comprehensive, cooperative experimental investigation for the primary purpose of studying the effects of varying the spacing of expansion joints in pavements containing closely spaced weakened-plane contraction joints. Incidental with this investigation, some questions arose regarding the ability of joints of the weakened-plane type to transfer loads and thus to control critical stresses caused by loads acting in their immediate vicinity. Previous research on the subject indicated that aggregate interlock as it occurred in such a joint separating two 20-foot slabs could not be depended upon to control load stresses (*5*).[1] For example the efficiency of the joint, as determined by strain measurements, was negligible in the winter when the maximum width of joint opening existed as a result of contraction of the abutting slabs. Furthermore, even in the summer when the width of opening was a minimum, the stress-reducing ability of the joint was questionable, varying from point to point along its edges due to the irregular and sloping manner in which the pavement fractured beneath the parting groove. Similar tests on a weakened-plane joint containing $\frac{3}{4}$-inch dowels spaced at 18-inch intervals showed a rather high average efficiency for all seasons of the year.

Since the tests just referred to were of limited scope and left unanswered certain questions, it was decided to include, as part of the cooperative investigation, a study of the structural efficiency of weakened-plane joints. Specifically, the research was planned to determine the structural behavior of joints of this type as affected by: (1) Type of coarse aggregate; (2) size of coarse aggregate; (3) presence or absence of dowel bars; (4) method of producing fracture at the joint; (5) compressive forces acting to close the joint; and (6) width of joint opening.

Earlier research on joints indicates that deflection relations are not a measure of the stress conditions that accompany them (*5*). Therefore, measurements of strain in the concrete are necessary to evaluate joints on the basis of their ability to reduce critical stresses caused by applied loads. Since strain measurements are time consuming and because the pavement at the test site should be protected from changes in temperature and moisture, this part of the investigation was conducted on test slabs that were not subjected to traffic. However, data obtained from the study should furnish exact information relative to the initial efficiency of similar weakened-plane joints incorporated in pavements in actual service. The influence of traffic and other factors acting over a period of time to change the initial effectiveness of joints of this type cannot, of course, be ascertained from these tests.

THE TEST SECTIONS DESCRIBED

The special tests were made by the Public Roads Administration on the concrete test pavement shown in figure 1. The pavement was divided into six sections, each being 30 feet in length, 20 feet in width, and of 8-inch uniform thickness. Each section was definitely separated from those adjoining it, and was divided longitudinally by a deformed metal center joint having $\frac{5}{8}$-inch diameter tie bars at 60-inch intervals and divided transversely by a weakened-plane joint of specific design. Thus, each section was subdivided into four panels, each 10 by 15 feet.

[1] Italic numbers in parentheses refer to the bibliography, p. 97.

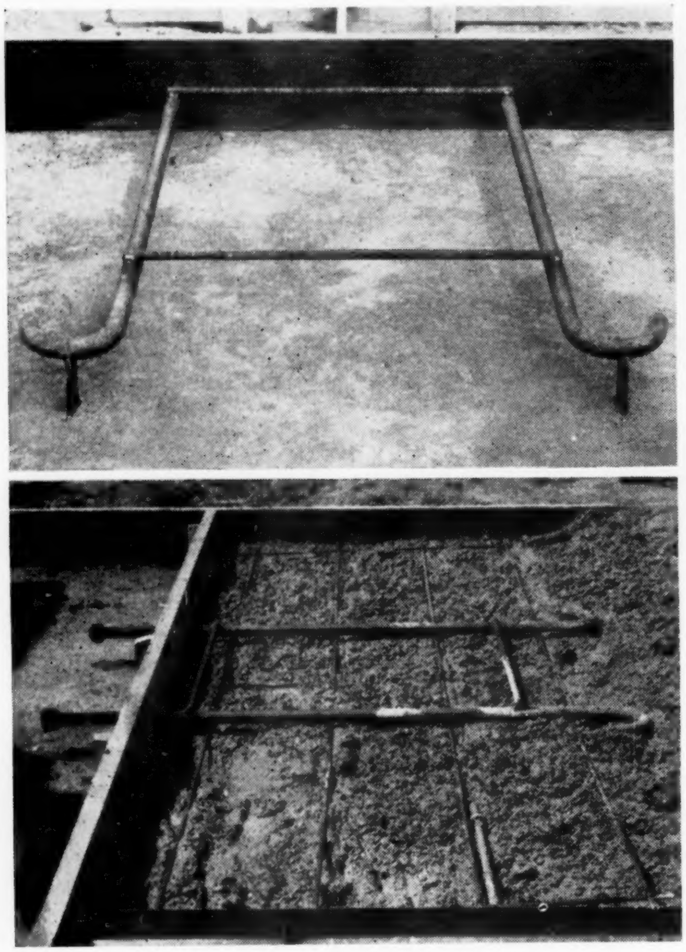


Figure 2.—Anchor-Bolt Assembly for Attachment of Apparatus Used in Controlling the Width of the Joint Opening.

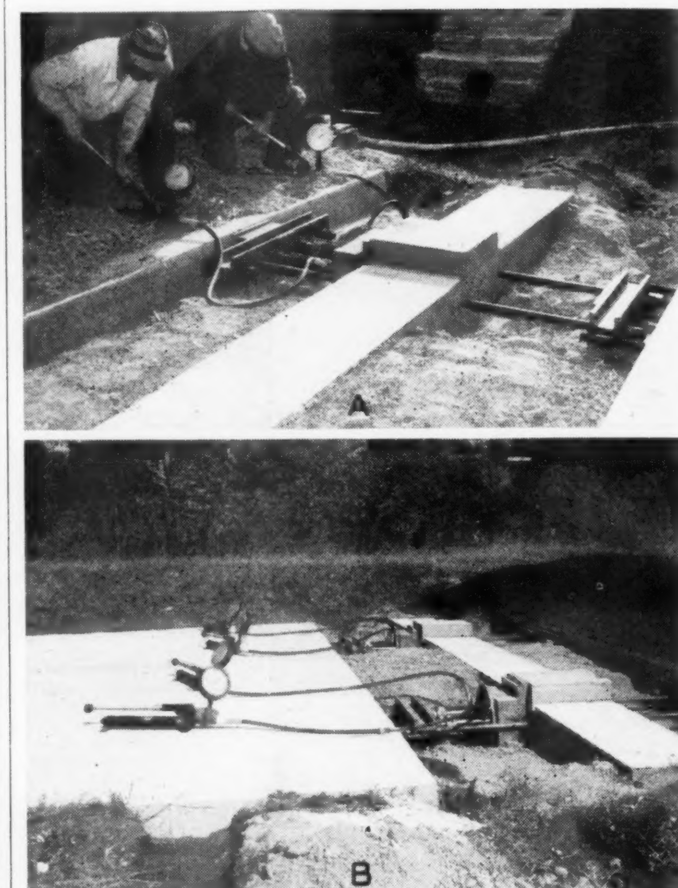


Figure 3.—Apparatus Used for Applying Compression and Controlling the Width of the Joint Opening. The Section Under Test Is Covered by Straw in Each Case, (*A*) Closing a Joint and (*B*) Opening a Joint.

At one end of every section, anchor-bolt assemblies, such as shown in figure 2, were embedded symmetrically in the concrete, one on each side of the longitudinal joint, to provide a means for attaching the apparatus used to control the opening of the weakened-plane joints. Between sections 1 and 2, 3 and 4, and 5 and 6 a concrete block or deadman was constructed to provide the reaction whenever direct compression was applied to the sections or whenever the joints were opened.

Figure 3 shows the reaction block together with the apparatus comprising the push-pull system. When closing a joint the jacks exerted pressure directly on the end of the section containing the joint. When opening a joint the jacks were moved to the other side of the reaction block and exerted tension in rods attached to the anchor-bolt assemblies of the section under test. The four hydraulic jacks used in the system were equipped with pressure gages in order to measure the forces applied to the section and, also, to insure uniform forces on both sides of the longitudinal joint.

The Subgrade.—The test sections were laid in the late summer of 1940 on the site of an earlier experimental concrete pavement (*4*). The soil underlying the earlier pavement was classified as a uniform brown silt loam (class A–4). For these later tests, the character of the subgrade immediately beneath the new sections was altered by thoroughly mixing 4 inches of clean sand with the top 4 inches of the original soil. After blending, the material was then compacted at optimum moisture to an average dry density of 136 pounds per cubic foot. This modification of the subgrade soil had the effect of increasing the value of the modulus of subgrade reaction by approximately 10 percent, as determined from load-deflection tests at the interior of the pavement.

The Concrete.—To investigate the influence of type of aggregate on the structural efficiency of the joints, two coarse aggregates having widely different characteristics were used. One aggregate was a siliceous gravel obtained from the Potomac River and the other was a crushed limestone obtained from near Frederick, Md. The gravel was typical of river gravels in that the edges of the individual pieces were well rounded and the surfaces were very smooth. The stone fragments had rather sharp angular fractures and relatively rough surfaces. The difference in character of the two aggregates is shown in figure 4.

The effect of aggregate size was studied for both types of coarse aggregate. This was accomplished by limiting the maximum size of coarse aggregate to 1 inch in some sections and to 2½ inches in others.

The program required the designing of four concrete mixes. For each, the cement was from the same source, the cement factor was approximately six sacks per cubic yard, and the fine aggregate was a Potomac River sand having a fineness modulus of 2.7. Table 1 shows the mix proportions, strength, and other properties of the concrete. The average slump in 17 tests was 2½ inches. Flexure tests were made on 8- by

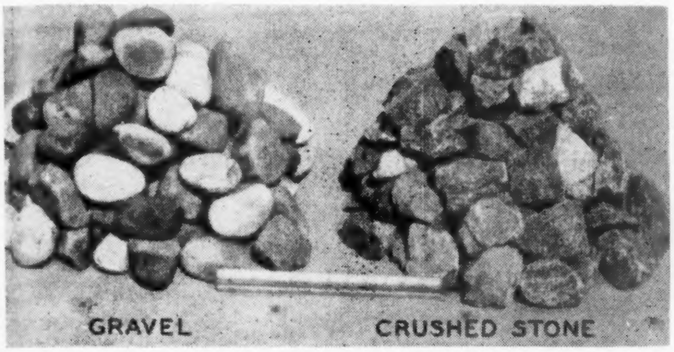


FIGURE 4.—LARGER SIZE OF COARSE AGGREGATE, 2½-INCH MAXIMUM SIZE.

TABLE 1.—*Mix characteristics and strength properties of the concrete*

Type of concrete	Coarse aggregate		Proportions by dry weight	Modulus of rupture		Crushing strength at 28 days	Modulus of elasticity at 16 months
	Type	Maximum size, (square opening)		28 days	17 months		
		Inches	*Pounds*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
I	Gravel	1	94 : 200 : 330	557	627	4, 620	4, 868, 000
II	do	2½	94 : 190 : 340	517	632	4, 850	4, 650, 000
III	Stone	1	94 : 232 : 308	611	725	4, 640	5, 407, 000
IV	do	2½	94 : 222 : 318	530	673	4, 520	5, 725, 000

TABLE 2.—*Features of the weakened-plane joints*

Joint No.	Type of joint (see fig. 5)	Coarse aggregate in adjacent slabs
1	C	Large gravel.
2	A	Small gravel.
3	A	Large gravel.
4	B	Small gravel.
5	A	Small crushed stone.
6	A	Large crushed stone.

8- by 40-inch beams, the load being applied at the third points of a 24-inch span. Compression tests were made on 6- by 12-inch cylinders. All specimens were cured in the same manner as the test sections (as nearly as possible) and, until a few days before testing, were stored in the ground adjacent to the pavement. The modulus of elasticity values were determined from 6- by 12-inch cylinders which had been stored in the ground for a period of 16 months.

THREE TYPES OF WEAKENED-PLANE JOINTS INVESTIGATED

Details of the three types of weakened-plane joints incorporated in the six sections are shown in figure 5. Types A and B are conventional joints. Type C is of special design having, in addition to a shallow surface groove, a bottom metal parting strip which is supposed to fracture the concrete in such a manner that load transfer will be developed to a more positive degree than is possible with the usual type of weakened-plane joint.[2] Figure 6 shows the bottom parting strip in place on the subgrade ready for the concrete to be cast.

Table 2 shows the features of each of the six weakened-plane joints. An examination of the table indicates certain comparisons in which only one variable is present. These are as follows:

[2] This joint was developed by Messrs. L. I. Hewes and William Bertwell.

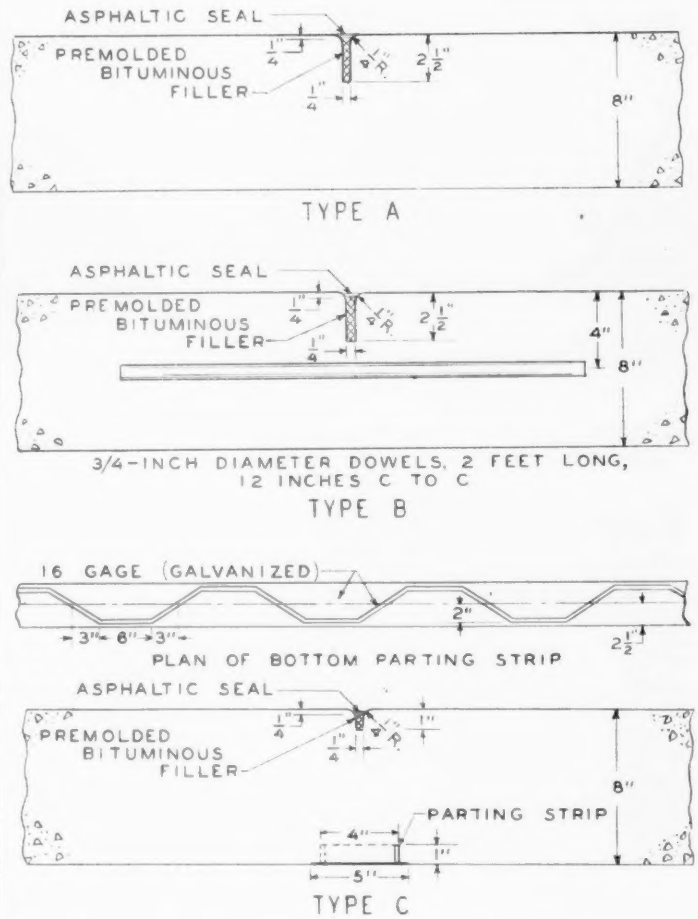


FIGURE 5.—DESIGNS OF WEAKENED-PLANE JOINTS INVESTIGATED.

Type of coarse aggregate:
- A. Small gravel versus small crushed stone (joint No. 2 versus joint No. 5).
- B. Large gravel versus large crushed stone (joint No. 3 versus joint No. 6).

Size of coarse aggregate:
- A. Small gravel versus large gravel (joint No. 2 versus joint No. 3).
- B. Small crushed stone versus large crushed stone (joint No. 5 versus joint No. 6).

Presence or absence of dowel bars:
- A. Joint No. 4 versus joint No. 2.

Method of fracturing pavement:
- A. Joint No. 1 versus joint No. 3.

LOADING AND TESTING PROGRAM SIMILAR TO THAT IN PREVIOUS STUDIES

The technique of testing to determine the efficiency of the various weakened-plane joints was similar to that used in previous studies. Simply, a load of given magnitude was applied successively at selected points at the interiors and free edges of the panels of a section containing a specific joint and at points along the edges of the joint. While this load was acting, the critical strain was measured in the upper surface of the slab. Critical strains occur directly under the contact area in the case of interior loading and, also, when a load is applied at the edges of a joint and at the free edges of a panel but away from the corners. For a load acting at or near the outside joint corners and the free corners of a panel, critical strains occur along the bisector of the corner angle at some distance from the load.

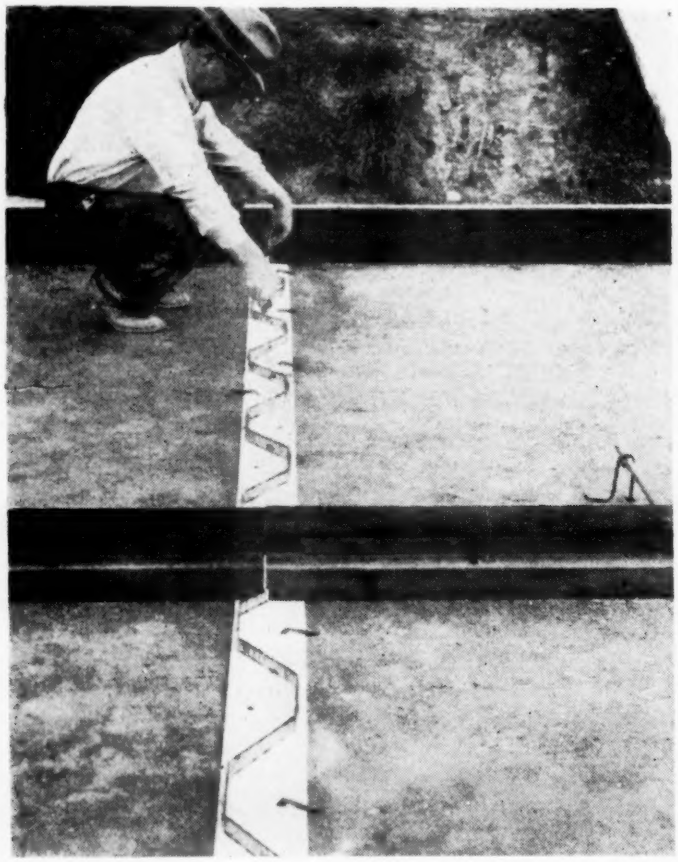


FIGURE 6.—BASE PLATE AND BOTTOM PARTING STRIP OF TYPE C JOINT IN PLACE ON THE SUBGRADE.

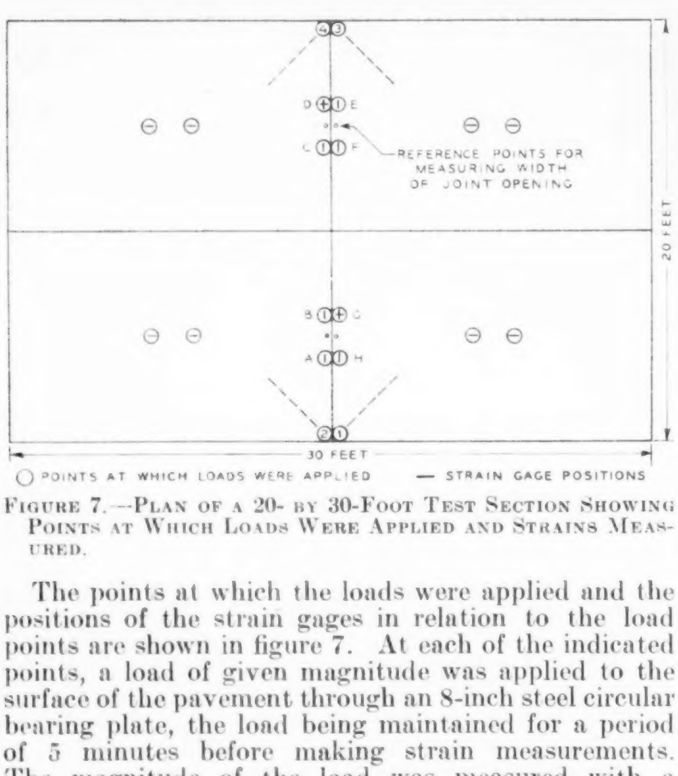


FIGURE 7.—PLAN OF A 20- BY 30-FOOT TEST SECTION SHOWING POINTS AT WHICH LOADS WERE APPLIED AND STRAINS MEASURED.

The points at which the loads were applied and the positions of the strain gages in relation to the load points are shown in figure 7. At each of the indicated points, a load of given magnitude was applied to the surface of the pavement through an 8-inch steel circular bearing plate, the load being maintained for a period of 5 minutes before making strain measurements. The magnitude of the load was measured with a dual-beam dynamometer. A sponge-rubber pad one-half inch in thickness was placed between the rigid plate and a previously prepared smooth surface. This surface was formed at each load point by seating the plate in a thin layer of plaster of paris and then removing the plate after the plaster had set. A large, cylindrical steel tank that transversely spanned the section provided, when partially filled with water, the necessary reaction for the vertical loads. Strains were measured at the positions indicated in the figure with a recording strain gage (*3*) installed between brass posts set into the upper surface of the concrete. The loading equipment and the method of measuring strains are described in detail in an earlier study of the structural behavior of concrete pavement slabs at the Arlington Experiment Farm (*4*).

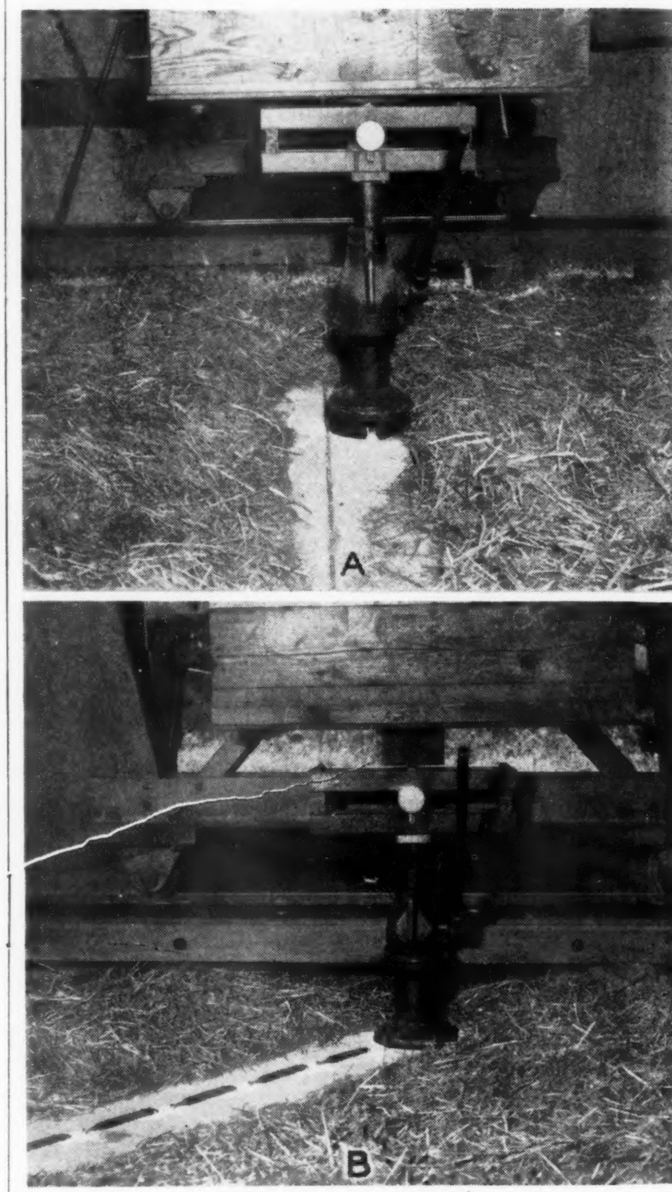


FIGURE 8.—APPARATUS USED FOR APPLYING THE LOAD AND THE ARRANGEMENT OF STRAIN GAGES IN RELATION TO THE APPLIED LOAD: (*A*) LOAD BEING APPLIED AT A JOINT EDGE POINT AND (*B*) LOAD BEING APPLIED AT A JOINT CORNER.

Figure 8 shows the load being applied and the strain gages in place at the joint edge (*A*) and the joint corner (*B*) of one of the test sections. In the case of edge loading, it will be noted that the bearing plate is grooved to accommodate the strain gage. The straw covering shown in the figure extended over the entire section and remained in place during the testing period. In addition, the section was sheltered from rain and direct sunlight. These protective measures minimized

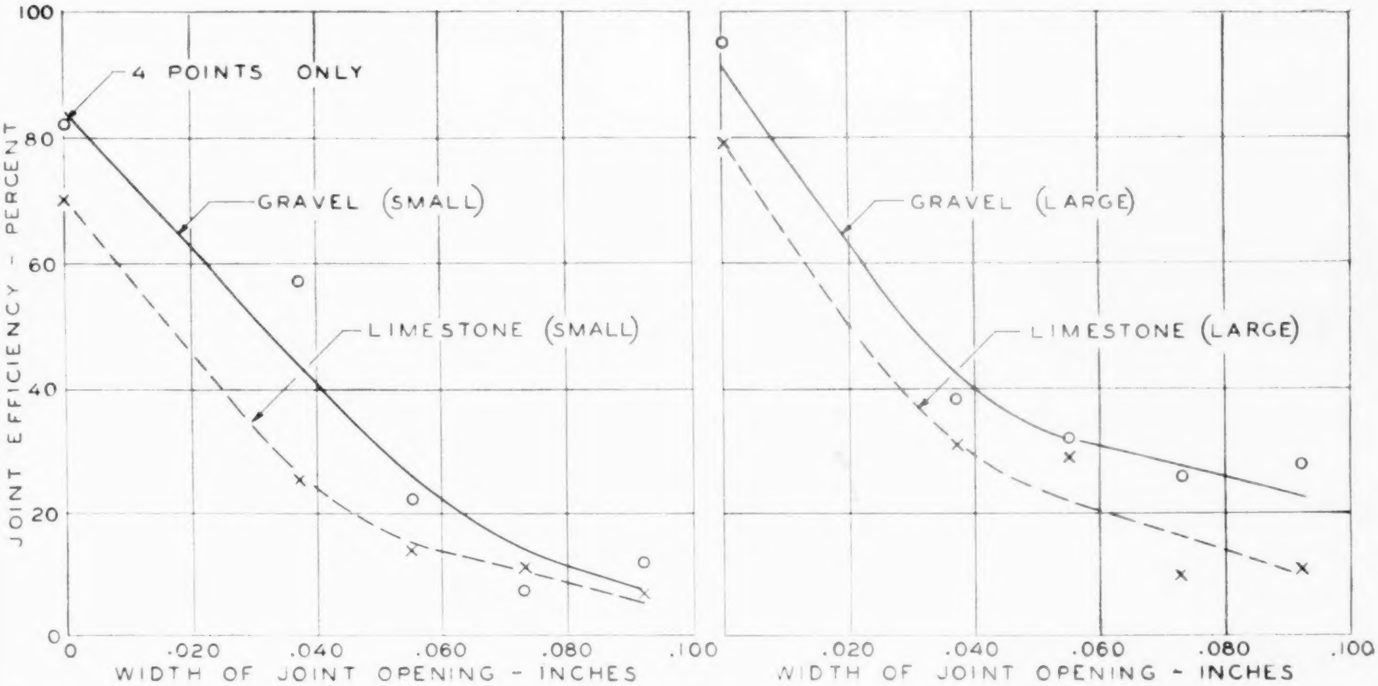


Figure 9.—Effect of Type of Coarse Aggregate on the Relation Between Joint Opening and Joint Efficiency as Determined With Edge Loading. Each Value Is an Average From Tests at Points A to H, Inclusive.

moisture changes in the subgrade and temperature warping of the slab.

TESTING SCHEDULE DESCRIBED

Testing of the various joints was conducted between June and October of 1941, 10 to 14 months after the pavement was laid. The testing schedule was essentially the same for each of the joints. Briefly, this schedule consisted of:

1. Measurement of the critical strains at the interiors of the four panels of a given section for a load of selected magnitude.
2. Application of a total compressive force of 120,000 pounds to one end of the section and, with the force acting, measurement of the critical strains at the joint edge points and at the joint corners.
3. Release of compressive force and measurement of the critical strains at the joint for successive controlled widths of opening of 0.037, 0.055, 0.073, 0.092, 0.110, and 0.220 inch. These are computed maximum joint openings which might occur during the annual cycle of temperature and moisture variations for slab lengths ranging from 10 to 60 feet. Initially, it was intended to measure strains both at the edge points and corners of the joints for each of the preceding openings; but, because of a time limit imposed by the necessity for vacating the site, the original schedule was curtailed. The schedule was reduced by obtaining strains at the edges of the joints for openings of 0.037, 0.055, 0.073, and 0.092 inch; and, with the exception of joint No. 2, at the corners for openings of 0.037, 0.073, 0.110, and 0.220 inch.
4. Opening of the joint to a width of one-half inch and measurement of the critical strains at the edge points and corners. For this opening all effects of aggregate interlocking had disappeared and the edges of the joint were acting as free slab ends. In the case of the doweled joint, all dowels were cut before testing at this opening.
5. Remeasurement of the critical strains at the interiors of the panels.

Before presenting the results of this study, attention is called to the fact that the zero widths of the joints are the widths established when the joints were closed by the 120,000-pound compressive force. Prior to the application of this force, it was observed that, with the exception of joint No. 1, the joints were opened about 0.015 inch. Under the compressive force the joints closed approximately 0.010 inch; thus, at the zero width there existed a residual opening of about 0.005 inch. The base readings at the zero joint width and subsequent determinations of the widths of joint opening were obtained with a vernier caliper by measuring between gage plugs cemented into the surface of the slab (see fig. 7).

EFFICIENCY OF THE JOINTS WITH A LOAD ACTING AT THE JOINT EDGE DETERMINED

Previous research has established that, for a slab of uniform thickness, the magnitude of the critical stress produced by a given load is minimum at the interior and maximum at the free edge. Consequently, in this investigation, the ability of the various joints to reduce stresses caused by a load acting at the joint edge, but away from the corners, will be determined by comparing the critical stress at the joint edge with that produced by the same load acting at the free end and at the interior of the slab.

From critical strains measured with the load acting at the aforementioned points, the efficiency of a joint may be computed by the following equation:

$$E=\frac{\sigma_f-\sigma_j}{\sigma_f-\sigma_i}$$

in which $E=$ joint efficiency;
$\sigma_f=$ critical stress for a given load applied at the free end;

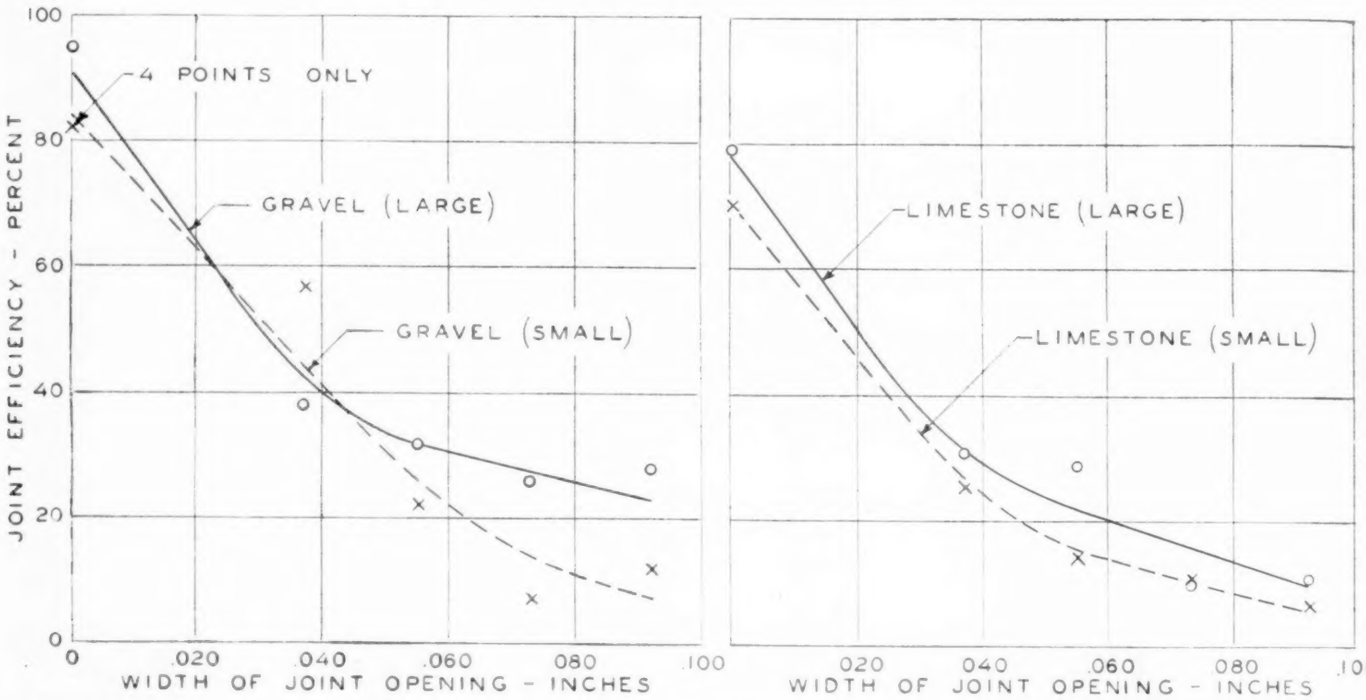
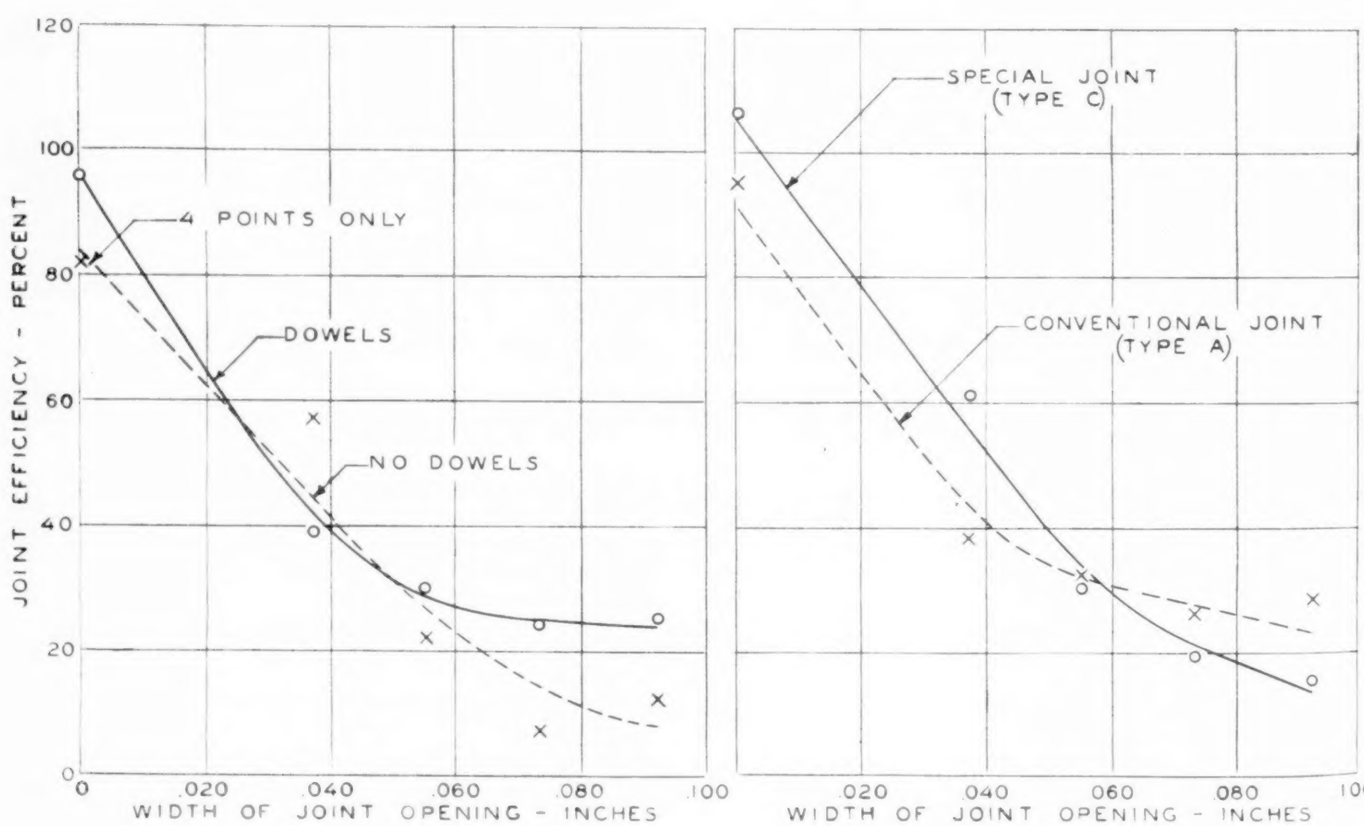


FIGURE 10.—EFFECT OF SIZE OF COARSE AGGREGATE ON THE RELATION BETWEEN JOINT OPENING AND JOINT EFFICIENCY AS DETERMINED WITH EDGE LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT POINTS A TO H, INCLUSIVE.



A - DOWELS VERSUS NO DOWELS (LOAD MIDWAY BETWEEN DOWELS) SMALL GRAVEL AGGREGATE

B - CONVENTIONAL VERSUS SPECIAL METHOD OF FRACTURING PAVEMENT LARGE GRAVEL AGGREGATE

FIGURE 11.—EFFECT OF DESIGN DETAILS ON THE RELATION BETWEEN JOINT OPENING AND JOINT EFFICIENCY AS DETERMINED WITH EDGE LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT POINTS A TO H, INCLUSIVE.

σ_j= critical stress for a given load applied at the joint edge;
σ_i= critical stress for a given load applied at the interior of the slab.

It will be noted that this equation indicates an efficiency of 100 percent if the critical stresses at the joint edge and at the interior are equal and, conversely, a joint efficiency of zero, if the critical stress at the joint edge is equal to that of the free end.

Figures 9, 10, and 11 show relations between edge efficiency and width of joint opening for each of the six weakened-plane joints listed in table 2. These relations are grouped in accordance with the comparisons given in connection with the discussion of table 2. Referring to the efficiency formula, the value of σ_i for a given section is based on the average of the strains measured (both before and after testing the joint) at the eight interior points of the slab units (see fig. 7). Similarly, the σ_j values and the value of σ_f for each joint are established from the strains measured successively at the eight edge points, A to H, inclusive, the strains for σ_f being measured after the joint had been pulled apart one-half inch.

The relations just presented show that efficiencies within the approximate range of 70 to 100 percent were attained by the various joints at the zero width of joint opening or, in other words, when the 120,000-pound compressive force was acting. If it is considered that the total force, used to close the joints, was uniformly distributed over the 5½-inch depth of the joint face below the parting grooves of joint types A and B, the average compression would be only 91 pounds per square inch. This value might even be reduced slightly if the subgrade resistance were taken into account. Hence, it appears that all of the weakened-plane joints studied in this investigation attained high efficiency in the presence of interfacial pressure much less than might develop in pavements under conditions of restrained expansion.

As previously mentioned, there was a residual joint opening of about 0.005 inch when the joints were tested while under compression. After completion of the regularly scheduled tests, joints Nos. 3 and 6 were closed and again tested while subjected to the 120,000-pound compressive force. Because of slight changes in the relative elevation of abutting slabs, the rough joint faces did not match as perfectly as before and the residual joint openings generally were much greater than those that existed at the time of the initial tests. At joint No. 3, a separation of 0.14 inch remained after application of the compressive force while at joint No. 6 the separation was 0.02 inch. It is of interest that the efficiencies of joints Nos. 3 and 6, which were initially 95 and 79 percent respectively, were still 61 and 49 percent respectively when retested under the rather severe conditions described above. These data indicate that weakened-plane joints when subjected to direct compressive forces are able to maintain fair efficiencies, even if the fractured faces are not perfectly matched.

Another observation made during the testing of the joints was that, in the presence of the 120,000-pound compressive force, the stress directly under the applied vertical load in a direction perpendicular to the edge of the joint was about one-half of the stress in a direction parallel to the edge of the joint. As the stress for a comparable load acting at a free end is virtually zero

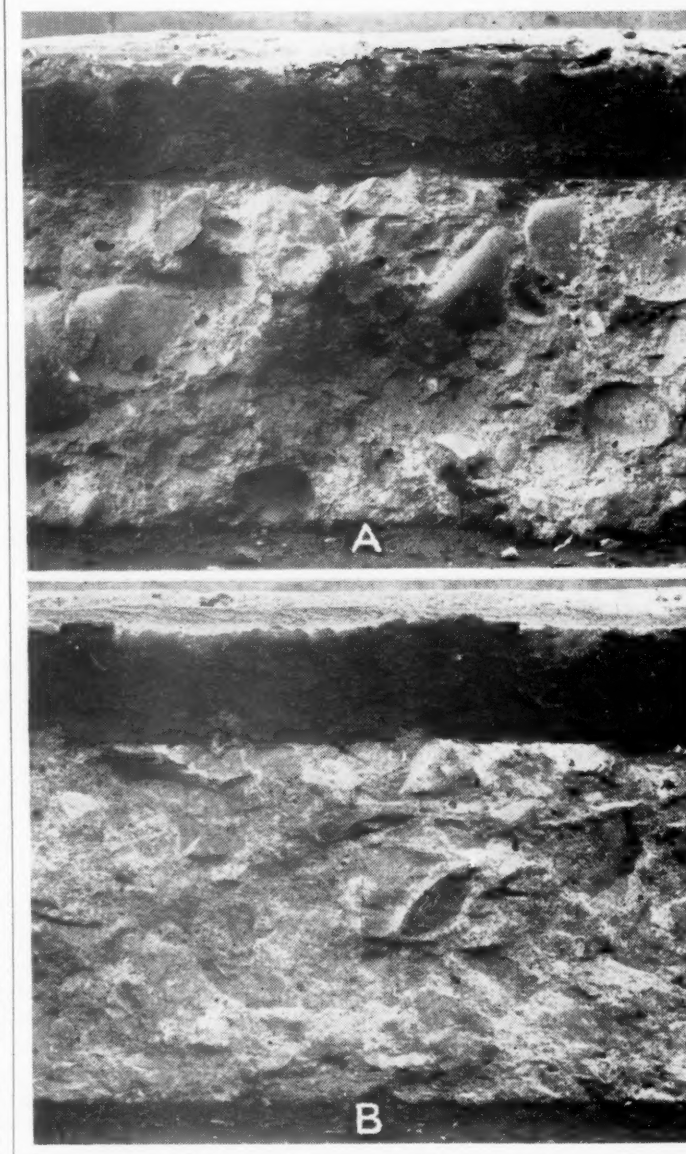


FIGURE 12.—EXPOSED FACES OF WEAKENED-PLANE JOINTS. (A) SECTION 3 CONTAINING LARGE GRAVEL COARSE AGGREGATE AND (B) SECTION 6 CONTAINING LARGE CRUSHED-STONE COARSE AGGREGATE.

in the perpendicular direction, the tests indicate that the compressive force makes it possible for the joint to transmit appreciable bending moment in addition to shear.

Again referring to the relations of figure 9, 10, and 11, it will be observed that the efficiencies of the joints decreased progressively as the width of joint opening was increased. At an opening of 0.037 inch, the efficiencies for the various joints, as established by the curves, ranged from 27 to 56 percent. At an opening of 0.092 inch, none of the joints had an efficiency greater than 25 percent.

JOINT EFFICIENCY INFLUENCED BY ROUGHNESS OF JOINT FACES

Figure 9 shows that the joints in sections containing the gravel coarse aggregate developed higher efficiencies than similar joints in sections having comparable sizes of the crushed-stone coarse aggregate. At some of the joint openings the difference was as much as 15 percent.

In an attempt to determine the cause of this difference in efficiency, a portion of one of the panels forming

FIGURE 13.—EXPOSED EDGE OF TYPE C WEAKENED-PLANE JOINT.

the joint was removed from each section so that the faces of the joints could be examined. This examination showed that in the sections having gravel coarse aggregate the joint faces were rougher than those of sections containing crushed-stone coarse aggregate due to the fact that when the crack occurred at the joints, the crushed-stone aggregate fractured whereas the gravel aggregate, in most instances, pulled out through loss of bond. This contrast in the surfaces of the fractured faces is shown in figure 12. The rough faces of the joints in sections containing gravel aggregate apparently were better able to transmit shear than were the relatively smooth faces of the joints in sections having crushed-stone aggregate.

In connection with the preceding discussion of the relative roughness of the joint faces, it is of interest to refer to the previously mentioned widths of opening that existed at joints Nos. 3 and 6 after they were closed upon completion of the regularly scheduled tests. It will be recalled that the residual opening of joint No. 3 (large gravel aggregate) was 0.14 inch but that of joint No. 6 (large crushed-stone aggregate) was only 0.02 inch.

Figure 10 offers a comparison between the efficiency relations of the two sizes of gravel aggregate and of the two sizes of crushed-stone aggregate. In the case of the gravel aggregate, the two efficiency curves are practically coincident for a width of joint opening up to 0.05 inch, after which the joint in the section containing the large-size gravel definitely was more efficient. This greater efficiency was probably due to the greater projections of the larger size of gravel aggregate. In the case of the crushed-stone aggregate, the joint in the section having the large-size stone was slightly, but only slightly, more efficient than a similar type of joint in the section built with the small-size stone. The probable reason for this condition is that both the large- and small-size crushed-stone aggregate fractured as the crack formed, thereby producing in both instances equally smooth joint faces.

In order to evaluate the effects of dowels on the relation between joint efficiency and joint opening, comparative efficiency data were established for two joints which were similar in every respect except that one contained dowels. These are shown in figure 11, *A*. Both of the joints were in sections having small gravel aggregate. Figure 5, shows that the dowels were three-fourths inch in diameter, 24 inches long, and were spaced at 12-inch intervals. The load was applied midway between dowels or at weakest locations along the joint edge. Any departure of the efficiency curve of the doweled joint from the curve of the undoweled joint is presumed to be caused by the action of the dowels.

The general coincidence of the two curves at openings between zero and 0.05 inch, inclusive, indicates that in this range the dowels aided but little the natural interlocking of the aggregate in the joint faces. However, after an opening of 0.05 inch the dowels had a marked effect on joint efficiency.

It will be recalled that earlier in this report mention was made of a previous study of a weakened-plane joint containing dowels spaced at 18-inch intervals. This study showed the joint to be 65 to 70 percent efficient for both summer and winter. As shown by the curve of figure 11, *A*, the doweled joint of the present investigation was only 25 percent efficient at openings greater than 0.06 inch. With our present knowledge derived from experimental studies of the structural behavior of dowels in transverse joints, this difference in efficiency cannot be fully explained. When more data are available a definite reason may be forthcoming. For instance, future research may shed some light upon the influence of subgrade stiffness and of pavement thickness on the structural performance of dowels. In the case of the two doweled joints in question, attention is called to the fact that the test pavement of the investigation being reported was not only constructed upon a somewhat more rigid subgrade; but was of 8-inch uniform thickness as compared to an approximate 7-inch thickness that existed at the point of test in the earlier study.

JOINT EFFICIENCY VARIES AT INDIVIDUAL POINTS ALONG JOINT EDGE

The efficiency relations of figure 11, *B* provide some measure of the effectiveness of the type C joint as compared with the conventional type of joint. Both of the joints in this comparison were in sections containing large gravel aggregate. The relations indicate that a clear-cut advantage of one type of joint over the other does not exist. At a joint opening of about 0.06 inch, both are equally efficient. At openings less than 0.06 inch, the special type of joint is more efficient than the conventional type; but at openings greater than 0.06 inch, the reverse is true.

Just prior to the testing of each of the joints, the earth shoulder was removed for the purpose of examining the crack that had formed. As remarked before, the crack that occurred at the special joint, No. 1, was unlike those that developed at the other five joints. Specifically, this crack was extremely fine and was visible for only one-half the depth of the slab; whereas, the cracks at the other joints were very definite, being open about 0.015 inch. Moreover, when the 120,000-

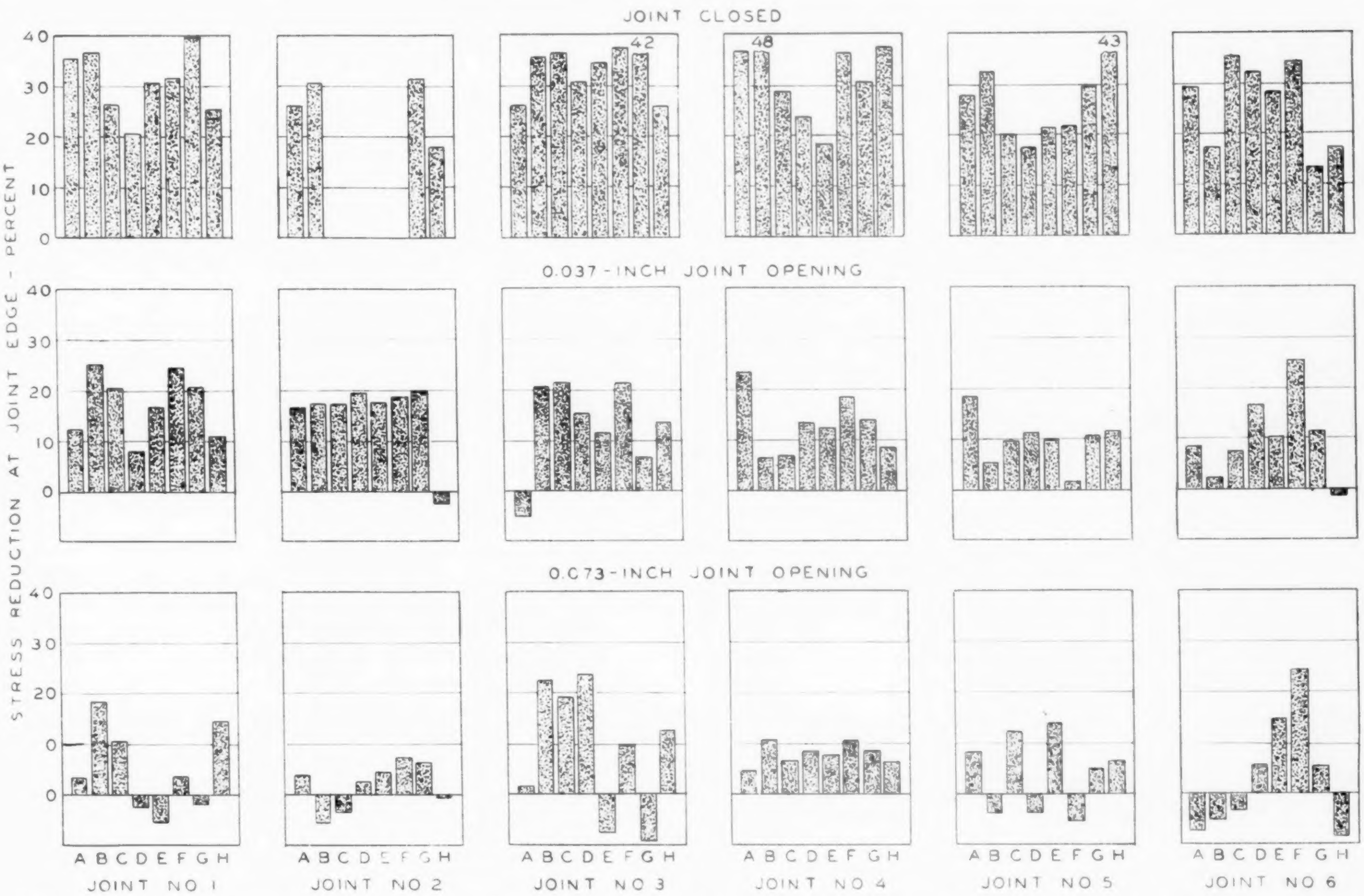


FIGURE 14. VARIATION IN THE AMOUNT OF STRESS REDUCTION AT INDIVIDUAL POINTS A TO H, INCLUSIVE, ALONG SLAB EDGES AT WEAKENED-PLANE JOINTS. CLOSED CONDITION IS THAT CREATED BY A COMPRESSIVE FORCE OF 120,000 POUNDS APPLIED AT ONE END OF THE SECTION.

pound compressive force was applied, the special joint closed only 0.003 inch as compared to a closure of 0.010 inch that was measured at the other joints. Besides the differences just mentioned, the special joint could be pulled apart only 0.006 inch, even by a total force of 110,000 pounds which, incidentally, started the 20- by 30-foot section to slip as a whole over the subgrade. As a consequence, wedges were used in conjunction with the pulling force to effect a complete separation.

The difference in the behavior of the special joint as compared with that of the other joints is an indication that the bottom parting strip failed to fracture the pavement as planned. Figure 13 shows the exposed joint edge of the special joint after a section of the pavement adjacent to the joint had been removed. This illustration shows rather conclusively that the bottom parting strip did not function in the intended manner.

The efficiencies of the various joints have thus far been discussed on the basis of relations determined from the averages of strains obtained at a number of load points. Figure 14 shows the amount of stress reduction found at individual points along the edges of the joints. These reduction values are expressed as percentages and are based on the stress determined at the points when the joint edges were acting as free slab ends. Since strains were measured at the same load point for all widths of joint opening and, also, for the basic free-end condition, the reduction values at the individual points, as shown in figure 14, should be affected but little by variations in the condition of the concrete and in subgrade support.

An estimate of the ability of the joints to reduce the critical stresses at the individual load points is provided by comparing their stress-reduction values with a similar value determined for the interiors of the sections. This interior stress-reduction value was found to be 33 percent, being established from the average of all of the critical strains obtained at the free ends and at the interiors of the panels of the six sections. Thus, if a reduction of 33 percent is attained at any point along the joint edge, then the joint at that point is assumed to be 100 percent efficient.

Referring to figure 14, it is observed that considerable variation in stress reduction was found at the individual points along the edges of the joints. Nevertheless, in spite of these variations, when the joints were closed and the compressive force acting, a relatively high degree of efficiency was developed at all of the points. At a joint opening of only 0.037 inch, however, the individual stress-reduction values indicate that aggregate interlock was not very dependable in reducing critical load stresses. When the joints were opened to a width of 0.073 inch, their stress-reducing ability diminished to such an extent that no reduction in stress was found at one-third of the points. In this respect all of the joints, with the exception of joint No. 4 which contains dowels, show a zero efficiency at two or more of the individual load points. The nonuni-

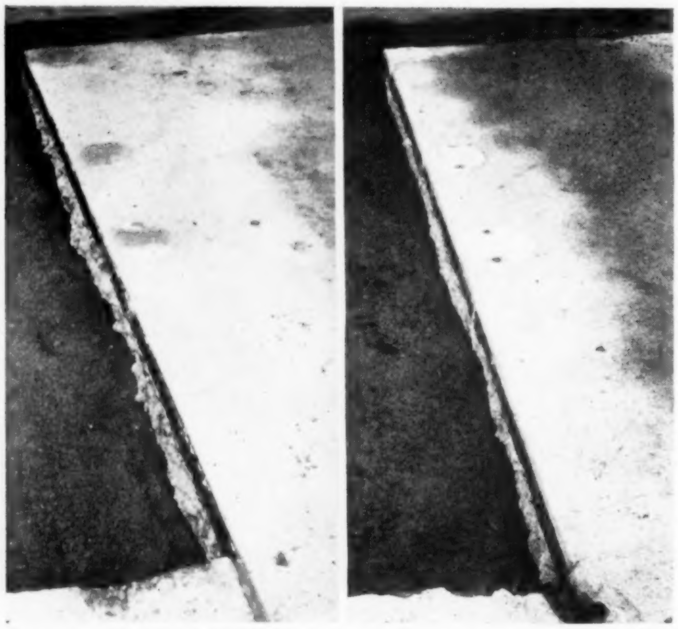


FIGURE 15.—EXPOSED EDGES OF TWO OF THE WEAKENED-PLANE JOINTS, SHOWING IRREGULAR LINE OF FRACTURE.

FIGURE 16.—EFFECT OF WIDTH OF JOINT OPENING ON THE MAGNITUDE AND DISTRIBUTION OF STRAIN ALONG THE BISECTOR OF THE CORNER ANGLE AS DETERMINED WITH CORNER LOADING. EACH VALUE IS AN AVERAGE FROM TESTS AT FOUR CORNERS OF JOINT NO 2.

formity of the stress-reduction values is not surprising when one considers the irregular manner in which cracks formed at joints of this type. Figure 15 shows the fractured surfaces at two of the test joints.

Summarizing the preceding data on the effectiveness of the various joints in controlling critical stresses originating from loads acting at some distance from the joint corners, it is concluded that: (1) in the presence of interfacial pressure, all of the weakened-plane joints were effective, even at relatively large residual joint openings; (2) without interfacial pressure, aggregate interlock was found to be an uncertain means of stress control regardless of the type and size of coarse aggregate; (3) the doweled joint showed a rather low average efficiency (about 25 percent) at widths of joint opening greater than 0.06 inch, but the presence of the dowels improved the uniformity of stress reduction at individual points along the edge of the joint; (4) in general, the rougher the joint face the greater the ability to reduce the critical load stress; and (5) roughness of joint faces depends in part upon whether or not the aggregate fractures or pulls out through loss of bond at the time the crack is formed.

EFFECTIVENESS OF THE JOINTS WITH A LOAD ACTING AT THE JOINT CORNER

The ability of the various weakened-plane joints to reduce critical stresses caused by loads acting at their edges has, heretofore, been confined to a discussion of the stresses that occur when the loads are applied at some distance from the joint corners. Under such a loading the critical stress is directly beneath the load and in a direction parallel to the joint edge. When loads are applied at or near the free corners of slabs of constant thickness, critical tensile stresses, as remarked before, develop in the upper surface of the slabs along the bisectors of the corner angles at some distance from the load.

As mentioned earlier, strains were measured along the bisectors of the outside corners of the six weakened-plane joints. These strains afford a means of determining how effectively the joints functioned in reducing the critical stresses for the condition of corner loading. Typical data obtained from tests at one of the joints are shown in figure 16. It is indicated that, for a given width of joint opening, the strains remain nearly constant over the distance between 16 and 32 inches from the corner, the maximum variation within these limits being less than 10 percent.

The data for zero width of joint opening were obtained with the joint under compression which explains why values for this condition are so much smaller than those found with the joint open a small amount.

The effect of type and size of coarse aggregate and of other variables on the reduction in the corner stresses at the various joint openings is given in figures 17, 18, and 19. Comparisons between the different joints in these three figures are the same as those previously described for the corresponding figures in the discussion of the edge condition of loading. The stress-reduction values shown in the graphs are expressed as percentages and were obtained by the following method: The average strains measured at the four corners of a given joint were first plotted in the manner shown in figure 16. From these curves the maximum average strain value for each joint opening was obtained. Using the maximum average strain value for a joint opening of 0.500 inch as a base, the amount of stress reduction effected by the joint at each of the lesser joint openings was then computed for each type of joint.

It is observed that the stress reduction at zero opening, or with the joints under compression, generally exceeded 50 percent of the stress for the free-corner condition. Theoretically, the maximum amount of

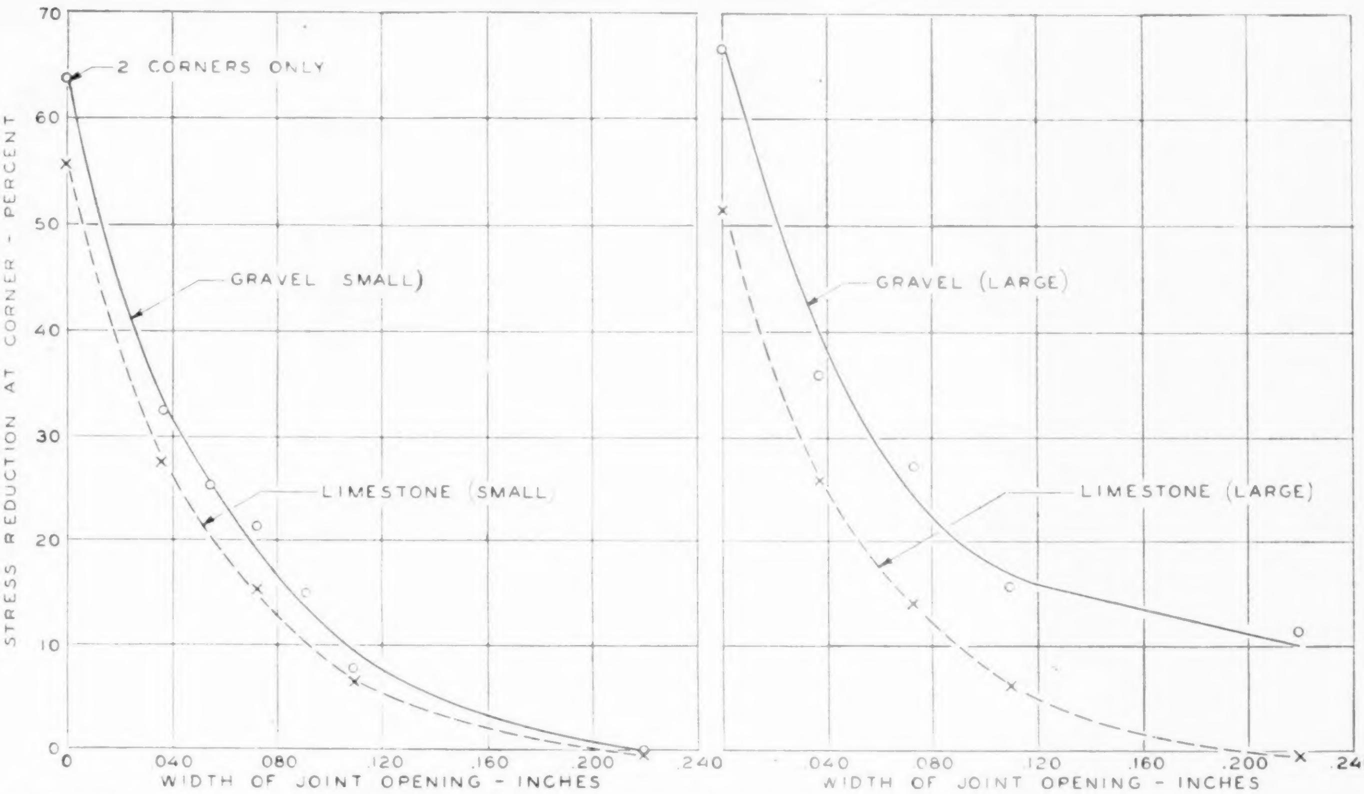


Figure 17.—Effect of Type of Coarse Aggregate on the Relation Between Joint Opening and Stress Reduction as Determined With Corner Loading. Each Value is Based on the Maximum Average Stress as Determined by Tests on Corners 1 to 4, Inclusive.

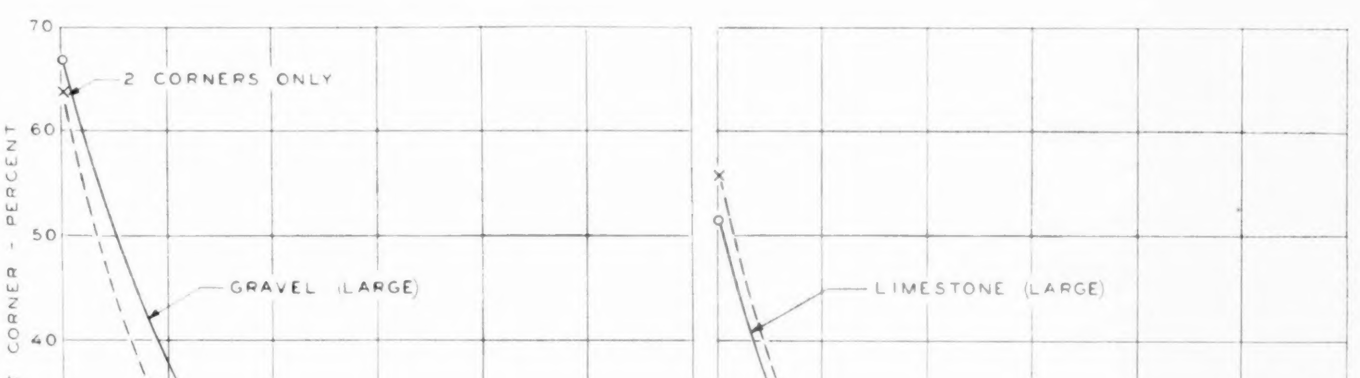


Figure 18.—Effect of Size of Coarse Aggregate on the Relation Between Joint Opening and Stress Reduction as Determined With Corner Loading. Each Value is Based on the Maximum Average Stress as Determined by Tests on Corners 1 to 4, Inclusive.

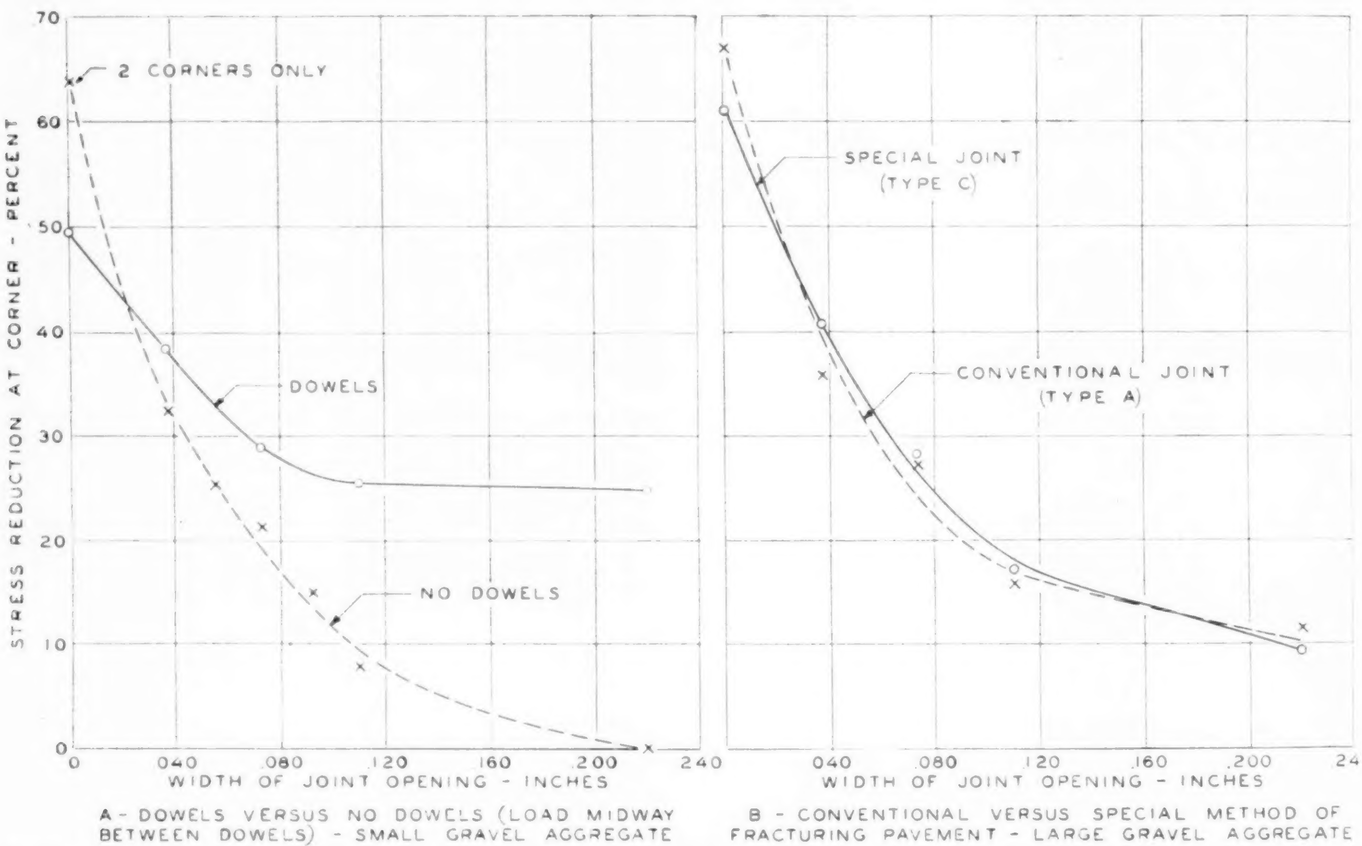


Figure 19.—Effect of Design Details on the Relation Between Joint Opening and Stress Reduction as Determined With Corner Loading. Each Value is Based on the Maximum Average Stress as Determined by Tests on Corners 1 to 4, Inclusive.

load that can be transferred by any joint is slightly less than 50 percent. Thus, with a load acting at a joint corner, the maximum reduction in stress that might be expected by load transfer is about 50 percent. However, if the joint is capable of exerting a moment which resists bending as well as transferring load through plain shear, it is possible that it might cause a reduction in stress greater than 50 percent of that for the free-corner condition. Apparently, the reductions in stress above 50 percent found at the zero width of joint opening were due to the resistance to bending caused by interfacial pressure at the joint.

The residual opening of joint No. 3 (large gravel aggregate) was 0.14 inch and that of joint No. 6 (large crushed-stone aggregate) was 0.02 inch when these joints were retested while under compression after completion of the scheduled tests. These separations had the effect of reducing the average corner stress-reduction value (joint under compression) of joint No. 6 from 67 to 25 percent and the value of joint No. 3 from 52 to 47 percent.

The relations presented in figures 17, 18, and 19 indicate that there was a sharp reduction in the effectiveness of the joints as the compression between the joint faces was released. Also that there was a progressive reduction in their effectiveness as the width of the joints were increased beyond 0.037 inch.

Figure 17 was prepared to show the effect of the type of coarse aggregate on the reduction in stress caused by aggregate interlock. It is indicated by this figure that the gravel gave a more effective interlock than the crushed stone. The difference between the two aggregate types was not great for the small-size coarse aggregate, but was appreciable in the case of the larger course aggregate.

In figure 18 is shown, for both types of coarse aggregate, the influence of the size of aggregate on the effectiveness of the aggregate interlock. It is observed that the large gravel gave a more effective interlock than the small gravel, but the size of the aggregate appeared to have little influence in the case of the crushed-stone aggregate.

It is believed that the reason for the more effective interlock shown by the gravel aggregate, particularly the large size, in these corner tests is the same as that discussed earlier in the presentation of the data for the edge condition of loading.

Comparisons are shown in figure 19 between similar weakened-plane joints with and without dowels and between the special weakened-plane joint, type C, and one of a conventional type. The coarse aggregate in the joints compared first (fig. 19, *A*) was small gravel while that in the other joints compared second (fig. 19, *B*) was large gravel.

It will be noted that at the zero width of joint opening (joint under compression), the stress reduction for the joint without dowels was greater than that of the joint with dowels. This may be the result of a difference in the effectiveness of the aggregate interlock in the two joints since the dowels have little or no opportunity to act while the joint is under compression or until some relative displacement between the two sides of the joint has developed. The stress reduction of the doweled joint exceeds that of the comparable un-

doweled joint at openings greater than approximately 0.02 inch, the difference being considerable at the large openings.

There appears to be little or no difference in the effectiveness of the aggregate interlock in the special weakened-plane joint, type C, and the conventional weakened-plane joint, type A.

EFFECTIVENESS OF AGGREGATE INTERLOCK INFLUENCED BY SMALL CHANGES IN JOINT WIDTH

The percentage of stress reduction at the four individual corners of each joint are shown for three different joint openings in figure 20. With the joint in a closed condition (under compression) the stress reduction values at the four corners were found to be quite uniform at all of the joints, although at joint No. 6 there was somewhat more variation than at the corners of the other five. However, as the joints were opened, appreciable variations were found in the amount of stress reduction among the four corners of each of the individual joints. At the 0.037-inch opening, the maximum variations occurred at joints Nos. 2, 5, and 6 where the stress-reduction values at the individual corners ranged as much as 25 percent and the amount of stress reduction at the corner showing the maximum was more than double that at the corner showing the minimum. The doweled joint No. 4 showed the most uniform stress reduction, the range in reduction values being approximately 5 percent.

At the 0.073-inch opening the stress reductions at some of the individual corners of several of the joints were so low as to indicate that, at this opening, aggregate interlock is not dependable. A possible exception to this is joint No. 3 with a large gravel aggregate. However, this joint would, without question, lose its effectiveness at only a small increase in opening or, perhaps, by the action of traffic gradually destroying the projecting aggregate.

The doweled joint at the 0.073-inch opening showed the greatest uniformity in percentages of stress reduction, 23 to 34 percent, and the highest average stress reduction, 29 percent. A special test was made of this joint at an opening of 1 inch and it was found that, at this opening, the stress reduction at the four corners ranged between 20 and 25 percent and averaged 22 percent. Thus, even at an opening of 1 inch, this joint not only maintained a high degree of uniformity in stress reduction but the difference in the average of the reduction values was only 7 percent when the width of the joint was changed from 0.073 to 1 inch. The efficiency of this doweled joint in controlling the corner stresses was not as high as might be desirable on the basis of perfect transmission of shear, but this was probably due to the fact that the dowels were not of sufficient diameter to be highly effective in an 8-inch pavement.

In the preceding discussion it was assumed that a joint acting in shear only should, if it is 100 percent effective, cause a reduction in the critical load stresses at the corners of 50 percent. It might be contended that it is only necessary for a joint to cause a sufficient reduction in the critical corner stresses to make them equal to those caused by an equivalent load acting at the interior of the slab.

A comparison of the average of all of the critical free-corner strains with the average of all of the critical interior strains obtained from the six sections of this study indicated that a reduction in the corner strains

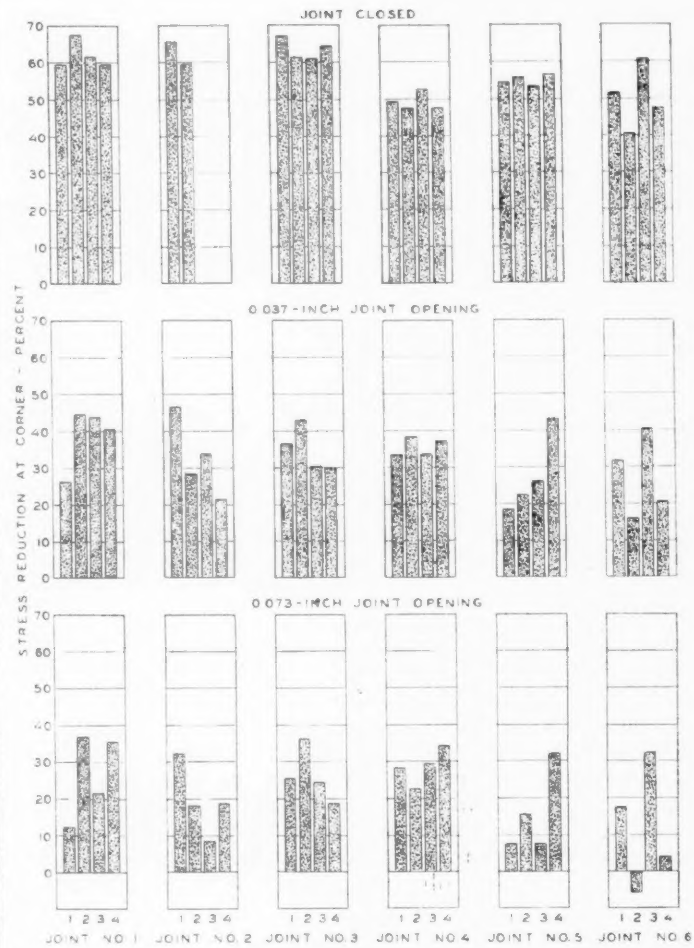


Figure 20.—Variation in the Amount of Stress Reduction at Individual Corners 1 to 4, Inclusive, at Weakened-Plane Joints. The Joint-Closed Condition is That Created by a Compressive Force of 120,000 Pounds Applied at One End of the Section.

of approximately 27 percent is necessary if the load stresses at the interior and corners of the slab are to be equalized. Thus, if it is desired to reduce the corner stresses only enough to make them approximately equal to the interior stresses, it might be considered that all of the joints with gravel aggregate were sufficiently effective at an opening of 0.037 inch. At this opening the stress reduction of a number of the individual corners with crushed-stone aggregate fell appreciably below 27 percent.

At the 0.073-inch opening the stress reduction at one or more of the individual corners was appreciably less than 27 percent in all of the joints except joint No. 4, which contained dowels.

It is concluded from the preceding data on the corner loading tests that: (1) on the whole, all of the joints were more effective in reducing critical corner stresses than they were in reducing critical edge stresses; (2) in the presence of interfacial pressure and small residual openings, all of the joints were not only extremely effective but a high degree of uniformity of stress reduction was indicated among the individual corners; (3) in the presence of interfacial pressure and a relatively large residual opening (0.14 inch) the reduction in the critical corner stress was sufficient to make the stress approximately equal to that caused by a load of the same magnitude acting at the interior;

(4) without interfacial pressure, all of the joints depending solely upon aggregate interlock to reduce critical stresses indicated a wide variation in the stress-reduction values; (5) the effectiveness of aggregate interlock was improved by roughness of the joint faces; and (6) the dowels improved the uniformity of stress reduction among the individual joint corners and, even at large openings, reduced the critical corner stress to a value approximately equal to that caused by a load of the same magnitude acting at the interior.

SIGNIFICANCE OF THE RESULTS

In concrete pavements the amount that closely spaced contraction joints may open during the winter when the concrete is in a contracted condition depends primarily upon the following factors: (1) The temperature of the concrete at the time it hardens; (2) the thermal coefficient of the concrete and the range in temperature to which the pavement is subjected; (3) the spacing of the joints; (4) the amount of shrinkage that occurs when the concrete hardens; and (5) the change in volume of the concrete due to seasonal moisture variation.

In a pavement laid without expansion joints, the expansion beyond the length at which it hardens is restrained; hence the maximum change in the widths of the contraction joints, due to volumetric changes in the concrete, is dependent upon the changes which occur from the time the pavement hardens to the time it reaches its minimum length. In a pavement with sufficient expansion space to permit full expansion, the maximum change in the width of the joints is dependent upon the volumetric changes during the annual cycle of moisture and temperature variations. Thus, to compute the maximum opening of the joints it is necessary in the case of pavements without expansion joints to assume the hardening temperature of the concrete and the minimum temperature that may occur. For pavements with expansion joints, it is necessary to assume the maximum and minimum temperatures that may occur in the concrete. It should be possible, with the help of published data on the temperature of concrete pavements, to assume these temperatures with reasonable accuracy. A number of determinations have been made of the thermal coefficient of concrete and while it varies with different aggregates it has been found to average approximately 0.000005 per degree Fahrenheit. Unless the spacing of the joints is known, it will be necessary to assume a value for purposes of computation.

There is a limited amount of data available on the shrinkage of concrete pavement slabs during the hardening period and, also, the change in length caused by seasonal moisture variations. In a study in Minnesota (*2*) measurements were made of the shrinkage in a number of slabs over a period of several years. It was found that during the early hardening period the amount of shrinkage was equivalent to that which would be caused by a reduction in the average temperature of the concrete of approximately 16° F., but that the amount of shrinkage increased appreciably later. Observations have been made of the changes in length of several full-size concrete pavement slabs from seasonal moisture variations. These observations indicate that the change in length depends upon the weather conditions, but is generally equivalent to that which might be caused by a change in the average temperature of the concrete of 20° to 30° F. Thus, within the limits of these data, it appears that the seasonal change in length of a pavement slab caused by seasonal moisture variation, from summer to winter, is approximately equal to the shrinkage which occurs when the concrete hardens.

To permit the measurement of the opening that is expected to occur later at contraction joints in concrete pavements, it is necessary to make the basic measurement before the joints fracture. This was done in the Michigan study (*1*). Measurements were made in the central part of a section of road without expansion joints and with contraction joints spaced 20 feet apart. During the second winter, that is, the first winter after the pavement had expanded to its maximum length, the average joint opening was 0.06 inch. The opening should be correspondingly greater or less for other contraction joint spacings.

EFFICIENCY OF WEAKENED-PLANE JOINTS MAY BE IMPAIRED WITH TIME

In pavements laid with expansion joints there is a tendency for contraction joints to open progressively until all available expansion space is dissipated. Because of this, the actual opening of the joints may, after a period of time, greatly exceed the computed openings or those observed during the first or second winter after the pavement is laid. In pavements without expansion joints there is little opportunity for progressive changes in the widths of the contraction joints. It has been observed, however, that where contraction joints are spaced at close intervals fracture may occur at some of the joints soon after the pavement is laid, yet be delayed as much as a year or more at others. Under these conditions some of the joints would open more and others correspondingly less than the normal amount. Should the open joints become filled with foreign matter they are apt to maintain a large part of this abnormal opening throughout the life of the pavement.

It was shown by the tests of the weakened-plane joints in this investigation that aggregate interlock was effective in stress control when the joints were closed or under compression, but that it was not dependable when the joints were open 0.037 inch or more, irrespective of the type or maximum size of the aggregate in the concrete. Thus, it must be concluded that aggregate interlock cannot be depended upon to give effective stress control throughout the full yearly temperature cycle in pavements with contraction joint spacings such that the joints can open an amount greater than approximately 0.04 inch.

Aggregate interlock may, during the early life of the pavement, be of some benefit in reducing the critical load stresses at weakened-plane joints which open less than this amount. The elimination of expansion joints might be expected to increase the length of the period over which the joints would retain their initial effectiveness, but it is doubtful if it would make them permanently effective. It has been found that joints which permitted a relative deflection, between the two sides of the joint, of approximately 0.008 inch or more under critical loads were ineffective in stress control; so that a joint to be effective must become fully engaged at very small deflections. Heavy, high-frequency traffic causes a severe hammering action in weakened-plane joints when they are open even small amounts and it is only natural to expect that, over a

period of several years, this hammering action would break down the aggregate interlock sufficiently to permit the small relative deflection that makes the joint ineffective in stress control.

In pavements where stress control at the joints is desirable it, therefore, appears necessary to use some more effective device for load transfer. The doweled joint tested in an open condition in this investigation, although more effective than the plain weakened-plane joints, did not show the structural efficiency expected. Other tests on doweled joints made by the Public Roads Administration indicate that ¾-inch dowels at 12-inch intervals are quite effective in stress control in slabs of 7 inches or less in thickness. This suggests that the size of the dowels should be progressively increased with increase in the thickness of the pavement.

PUMPING AND FAULTING AT CONTRACTION JOINTS DISCUSSED

One of the most serious conditions that has developed in concrete pavements in recent years is faulting and pumping at expansion joints, open cracks, and open weakened-plane joints. This condition is probably aggravated by a greater frequency of heavy loads on the pavements during the war period than has been common in the past. Faulting generally follows pumping, but may occur without pumping. The type of faulting not associated with pumping has been found on subgrades of both fine-grained and granular soils, but seems to be most common and serious with silt and clay subgrade soils. Pumping is definitely associated with fine-grained subgrade soils and observations indicate that the water which is pumped to the surface from beneath the slabs is largely surface water that has leaked through the joints, open cracks, and along the edges of the pavement. Hence, defective sealing of joints and cracks is a condition contributing to pumping and the type of faulting associated with it.

Aggregate interlock at weakened-plane joints and cracks that are not permitted to open too widely seems to have been generally effective in preventing faulting of the type not associated with pumping. In pavements laid without expansion joints and closely spaced contraction joints (15 to 20 feet) aggregate interlock probably will be effective in preventing serious faulting of this type. If conditions are such as to permit an appreciable progressive opening of the weakened-plane joints, however, it is thought that an additional means for load transfer should be provided.

Adequate load transfer acts in two ways to prevent or reduce the amount of pumping and the accompanying faulting at joints in concrete pavements. First, it reduces the total deflection that occurs when the loads pass over the joint, thus causing a reduction in the amount of water and soil pumped to the surface with each repetition of load. Second, it reduces the relative deflections of the two slab ends when a load passes over the joint thus helping to preserve the seal of the joints and prevent the infiltration of surface water. Tests have shown that the total deflections at joints may be reduced nearly 50 percent and the relative deflections nearly 100 percent by adequate load transfer installations.

It has been found that aggregate interlock may be very helpful in reducing the total deflections and relative deflections at joints if they are not allowed to open too widely. However, it must be recognized that heavy, high-frequency traffic may in time seriously reduce the effectiveness of aggregate interlock. From load-deflection studies on slabs not subjected to traffic, it has been observed that the performance of weakened-plane joints with dowels was definitely better than those without dowels. Also, it may be expected that the doweled joints will retain their effectiveness at wider openings and over a longer period of time.

The effectiveness of load-transfer devices in preventing pumping and the accompanying faulting naturally varies with the character of the devices. Field observations of pavements indicate that, in many instances, where the conditions are not too severe, plain round ¾-inch dowels spaced at close intervals have been effective in preventing or minimizing pumping and faulting. These same observations indicate that all types of load-transfer devices are not equally effective for this purpose.

Since aggregate interlock is of definite value in preventing or reducing the amount of pumping and faulting in weakened-plane joints when they are not allowed to open too widely, it is desirable that some provision should be made in the design of the pavement to limit the opening of the joints. This involves the adjustment of the spacing of such joints to limit the magnitude of the seasonal opening from temperature and moisture variations in the concrete, and the limiting of available expansion space to prevent appreciable progressive opening of the intermediate joints, either by actually eliminating the expansion space or by using an expansion joint filler that offers considerable resistance to compression.

It is desirable that further studies be made to determine the effectiveness of aggregate interlock and other methods of load transfer in preventing pumping and faulting and the conditions under which other corrective measures may be necessary.

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DISINTEGRATION OF BRIDGE CONCRETE IN THE WEST

Reported by F. H. JACKSON, Principal Engineer of Tests, Division of Physical Research, Public Roads Administration[1]

A report giving the results of an inspection of numerous structures located in certain areas of Wyoming, Oregon, Washington, and California, made in connection with a study of the causes of the disintegration of concrete in these areas and including suggestions for revisions in specifications to govern future work.

FOR SOME YEARS it has been evident that concrete bridges located in certain areas of the west are not giving as satisfactory service insofar as the durability of the concrete is concerned as similar structures in other areas built under identical specifications and subject, in general, to the same degree of engineering supervision during construction. Reports by representatives of the Public Roads Administration, as early as 1935, called attention to disintegration which was starting on some of the bridges in the Jackson Hole area just south of Yellowstone Park. These bridges were at that time only about 4 years old. They now show evidence of advanced disintegration. Moreover, other bridges much more recently constructed and located within the park are showing initial evidence of distress of about the same type as was first noted on the older bridges.

Mr. G. S. Paxson, bridge engineer of the Oregon State Highway Commission, in a letter dated June 17, 1943, described several distinct types of disintegration of concrete which had developed about the same time (1934–35) on certain bridges located in eastern Oregon, whereas bridges of the same design and built under the same specifications, but located in the western part of the State, were showing no signs of trouble. Mr. Bailey Tremper, materials engineer, Washington Department of Highways, has also called attention to evidences of disintegration of concrete in certain areas of that State. In a paper published in June 1941 (*6*)[2] he discussed deterioration of concrete in eastern Washington and in the general area of Mount Rainier National Park. The trouble in the park area was identified by Tremper as probably due to a cement alkali-aggregate reaction, a particular type of concrete failure which Mr. T. F. Stanton, materials and research engineer for the California Division of Highways described about 1 year earlier as prevalent in certain areas in southern California (*4*). Numerous articles on the subject have appeared since.

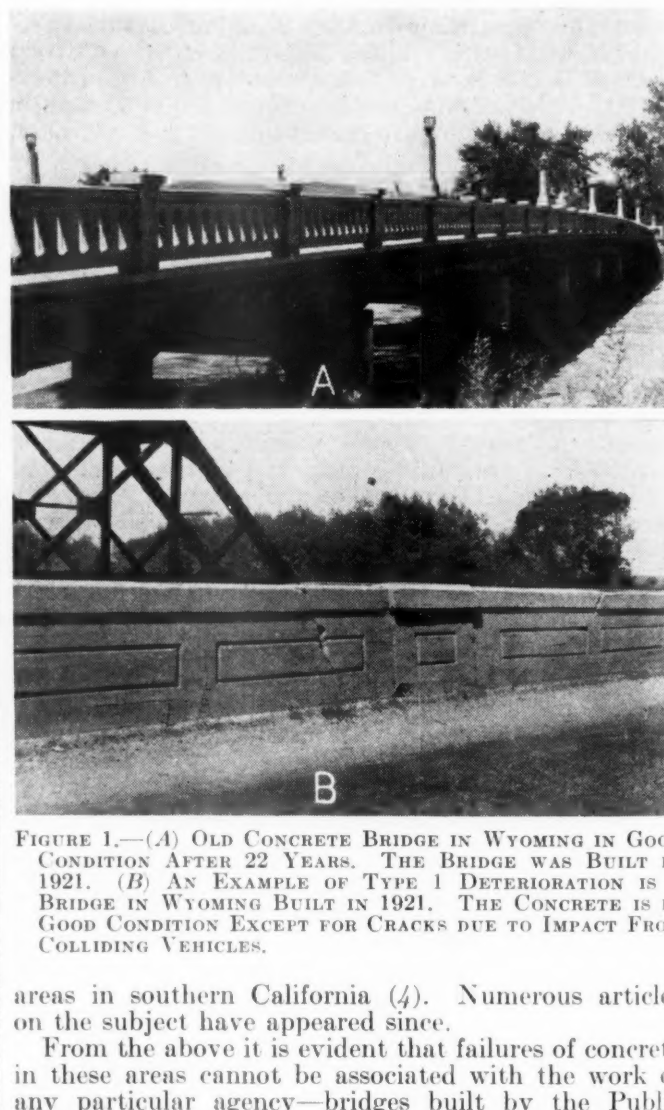


FIGURE 1.—(*A*) OLD CONCRETE BRIDGE IN WYOMING IN GOOD CONDITION AFTER 22 YEARS. THE BRIDGE WAS BUILT IN 1921. (*B*) AN EXAMPLE OF TYPE 1 DETERIORATION IS A BRIDGE IN WYOMING BUILT IN 1921. THE CONCRETE IS IN GOOD CONDITION EXCEPT FOR CRACKS DUE TO IMPACT FROM COLLIDING VEHICLES.

From the above it is evident that failures of concrete in these areas cannot be associated with the work of any particular agency—bridges built by the Public Roads Administration as well as those constructed by the States showed evidence of distress under certain conditions. Furthermore, it should be emphasized that even in the areas where trouble has occurred not all of the bridges are showing distress. There are many good bridges in these areas, particularly among the older structures (fig. 1, *A*).[3] However, the defects which have developed, in some cases after only 3 or 4 years, are sufficiently serious to warrant a very intensive search for the causes in order that steps may be taken to avoid these troubles in the future.

[1] Three separate parties were organized to make these inspections. In addition to the writer, they included the following:

(1) For the inspections at Kimball, Nebr., and in Wyoming including Yellowstone Park: B. W. Matteson, district engineer, District 3, Denver; H. R. Angwin, senior highway bridge engineer, Regional Office, San Francisco; A. V. Williamson, senior highway engineer, District 3, Denver; W. D. Ross, materials engineer, District 3, Denver; H. S. Meissner, engineer in charge cement laboratory, Bureau of Reclamation, Denver, and M. A. Ver Brugge, materials engineer, Wyoming State Highway Department.

E. H. Cowan, highway engineer, District 3, in charge of Public Roads work in Yellowstone Park, accompanied the party through the park. I. E. Russell, chief materials engineer, Wyoming, accompanied the party to Kimball, Nebr., in place of Ver Brugge.

(2) For the inspections in Oregon and Washington: G. S. Paxson, bridge engineer, Oregon State Highway Commission, Salem; Bailey Tremper, materials engineer, Washington State Department of Highways, Olympia; H. R. Angwin, senior highway bridge engineer, Regional Office, San Francisco, and R. M. Schwegler, materials engineer, District 1, Portland.

Mr. Stephenson, assistant bridge engineer, Oregon, accompanied the party during the inspections in Oregon only.

(3) For the inspections in California: T. E. Stanton, materials and research engineer, California Division of Highways, Sacramento; Fred Klein, senior highway bridge engineer, Regional Office, San Francisco, and D. J. Steele, materials engineer, District 2, San Francisco.

The itinerary for Wyoming and the Yellowstone Park was arranged by Matteson, for Oregon and Washington by Paxson and Tremper, and for California by Stanton. Inspections in Wyoming and the Yellowstone Park were made during the period June 12 to 20, inclusive; those in Oregon and Washington, June 23 to 30, inclusive; and those in California, July 4 to 11, inclusive, all in 1944.

[2] Italic numbers in parentheses refer to the bibliography, p. 111.

[3] The photographs accompanying this report were selected to illustrate as clearly as possible the various types of failure that were observed during the inspection. In addition to those taken by the writer, photographs taken by the following members of the inspection parties are included: Messrs. W. D. Ross, A. V. Williamson, H. S. Meissner, Bailey Tremper, T. E. Stanton, and R. M. Schwegler.

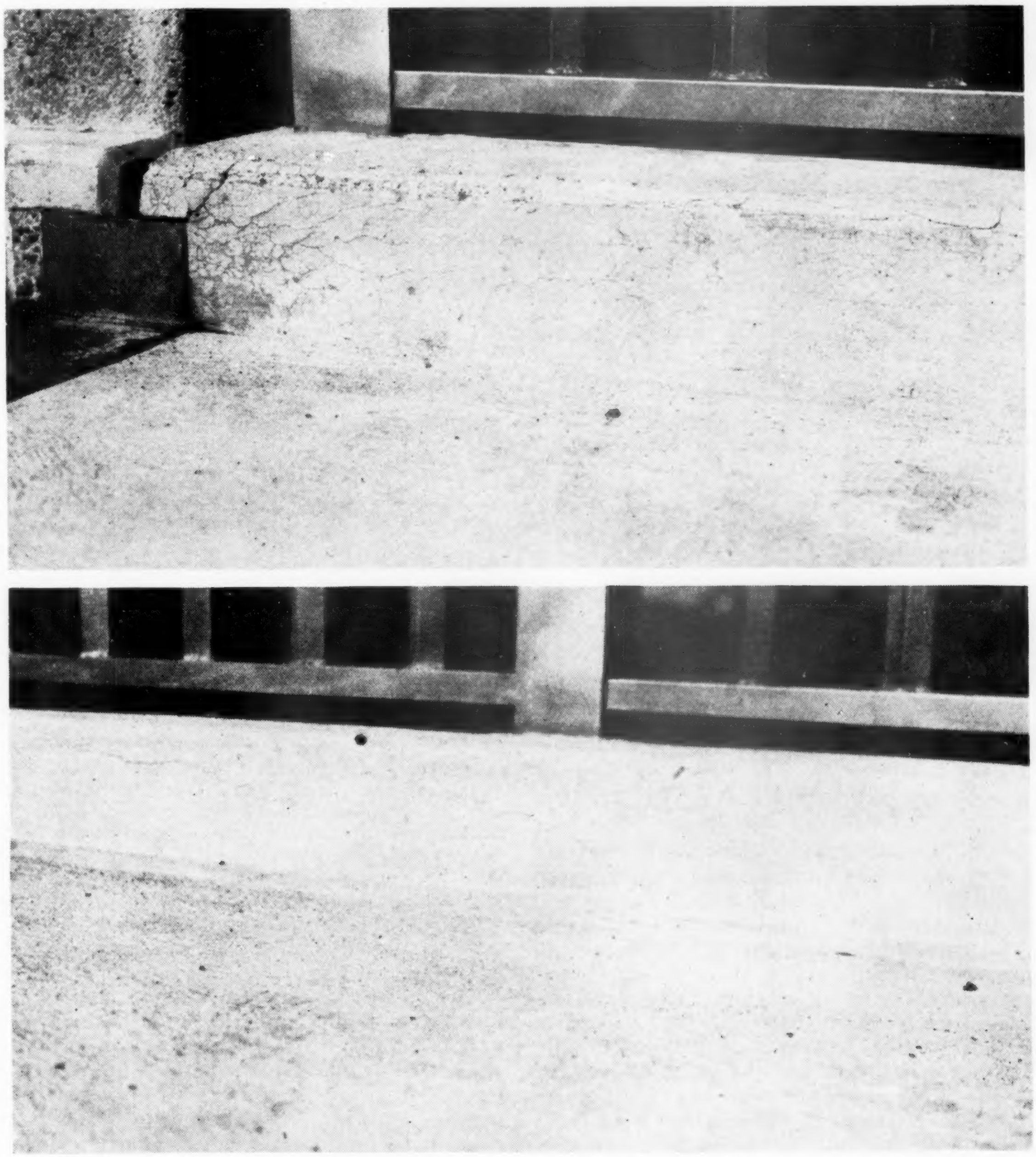


FIGURE 2.—FIRST STAGE OF TYPE 2 DETERIORATION ON A BRIDGE IN YELLOWSTONE PARK. FINE CRACKS APPEAR AT JOINT AND ALONG CURB.

The writer was assigned to make an inspection of certain bridges in and near Yellowstone Park and to confer with field engineers regarding the problem. Since the Public Roads Administration is actively co-operating with both Oregon and Washington in an effort to determine the causes of concrete failures in the Pacific Northwest, it was felt that the inspections should be extended to include bridges in these States. Arrangements to this end were made as well as arrangements to visit a number of structures in southern California which show evidence of failure due to cement alkali-aggregate reaction.

It is believed that because of the similarity of the problems the observations made during the inspection of structures in Oregon, Washington, and California, as well as the conclusion reached and corrective measures proposed, can appropriately be included in the report on the Yellowstone Park problem. In the following discussion it is proposed first, to describe the types of failure observed on the trip; second, to summarize the observations made in each of the areas visited; third, to discuss certain specific matters which may have a bearing on the problem, including construction variables, modern versus old-fashioned cements, air entrainment, alkalies, etc., and fourth, to indicate the corrective measures which should be adopted.

FOUR TYPES OF DETERIORATION OBSERVED

During the course of these inspections several distinct types of concrete deterioration were observed. These may be classified roughly as follows:

Type 1. Deterioration due to gradual or normal weathering.—This is indicated usually by slight surface erosion and pitting, rounded corners, etc. Many old structures also show cracks due to settlement and

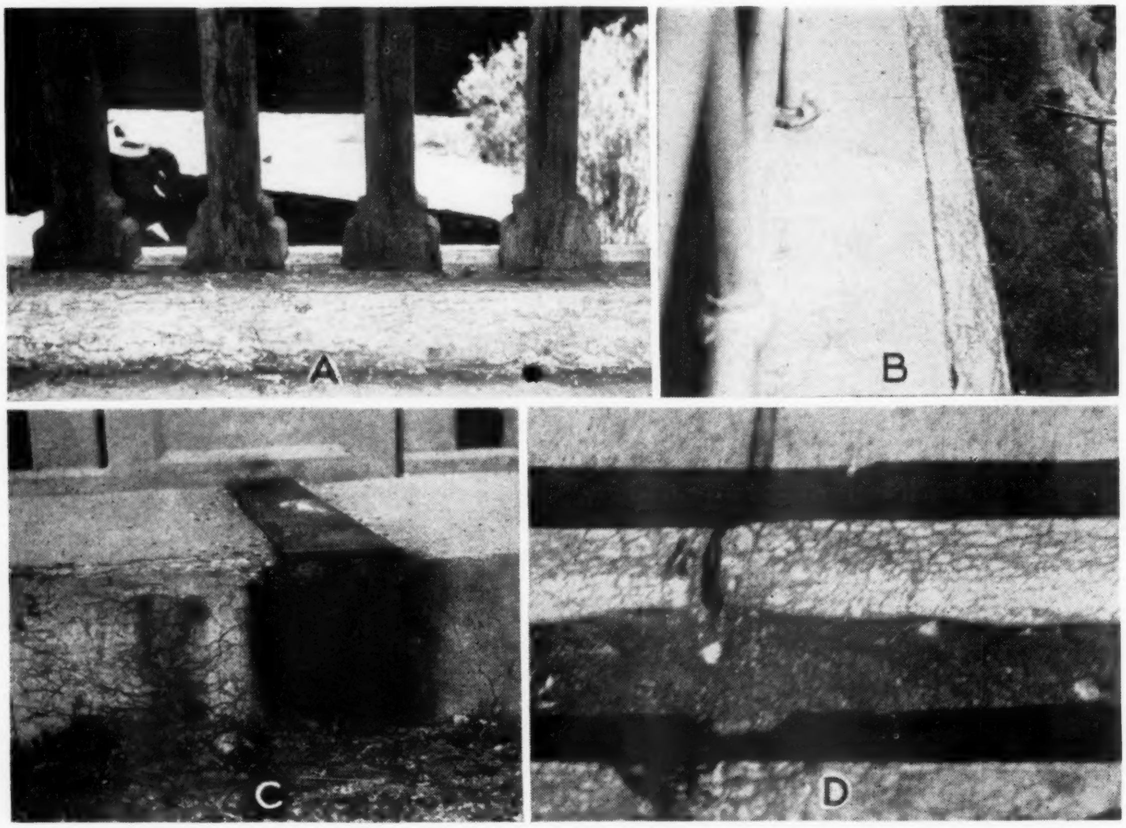


FIGURE 3.—EXAMPLES OF INTERMEDIATE STAGE OF TYPE 2 DETERIORATION IN (*A*) OREGON, (*B*) WYOMING, (*C*) Illinois, and (*D*) Wyoming. THE CRACKS ARE FILLED WITH DARK DEPOSIT SOMETIMES CALLED D-LINES.

impact from colliding vehicles (fig. 1, *B*). The concrete has a good ring under the hammer and a good sharp chip may be obtained. This condition is evidence of durable concrete, especially when found on structures subject to severe natural weathering. Concrete of this character was noted on many of the older bridges.

Type 2. Deterioration due to accelerated weathering.—This type of failure is all too common, particularly in areas subject to severe frost action. It is usually evidenced first by the formation of fine cracks on the surface of exposed members such as curbs, handrails, end posts, tops of retaining walls, wing walls, etc. They usually appear first on surfaces adjacent to construction joints or at cracks and other points where water can enter (fig. 2, *A*). They also tend to form along the edges of thin members such as curbs and handrails (fig. 2, *B*). The cracks are ordinarily close to and parallel to the edge and are usually filled with a dark-colored deposit, probably calcium carbonate (fig. 3). Concrete so affected has little strength and can be broken easily under the hammer. The matrix has a dull chalky appearance in sharp contrast to the dense, compact, bluish-gray matrix usually found in good concrete. Progressive and rapid disintegration usually follows the appearance of these D-lines as they are frequently called.

As might be expected distress of the type 2 variety, if it develops at all, usually starts in the relatively thin members of the superstructure. They are the members most directly exposed to weathering. The sections are frequently thin, necessitating the use of a rather fine graded, coarse aggregate which, in connection with the fact that these members usually contain considerable reinforcing steel, tends to result in the use of a higher water content than in the heavier members, even though the cement content in bags per cubic yard may be exactly the same. This raises a question as to whether thin sections should be used where the concrete will be subjected to severe weathering.

D-line cracking of the above type should not be confused with the familiar crazing cracks which often appear on highly finished smooth surfaces and which are, no doubt, due to surface shrinkage. D-lines on the other hand are probably the result of internal expansion resulting from failure due to alternate freezing and thawing. So far as the writer is aware, they are not found in areas where freezing does not occur.

Type 3. Deterioration due to salt scaling.—This type of surface defect is the result of using calcium or sodium chloride for ice removal. Pavements and sidewalks so treated, especially if the concrete is new, will almost invariably scale badly, the mortar surface being, in many cases, completely removed during the first winter (fig. 4). A specific remedy for this condition has been found in air entrainment, which will be discussed later.

Type 4. Cracking due to abnormal expansion.—All concrete will expand or contract when subjected to changes in either temperature or moisture content. This property is recognized and provided for in the design of structures. Occasionally, however, a tendency towards abnormal expansion develops which cannot possibly be explained in the usual way. Failures due to abnormal expansion are characterized by the formation of relatively wide, open cracks of appreciable depth and usually roughly parallel to the longitudinal axis (fig. 5). Random or pattern cracks, also open and rather widely spaced, are frequently formed (fig. 6). Failure due to abnormal expansion may or may not be

FIGURE 4.—AN EXAMPLE OF TYPE 3 DETERIORATION ON A BRIDGE IN WYOMING, SHOWING SURFACE SCALING DUE TO THE USE OF SALT FOR ICE REMOVAL.

followed by ordinary weathering (type 2) depending upon the severity of exposure. For example, certain structures in southern California which showed characteristic cracking of this type as early as 1937 are still in about the same condition, indicating that expansion has ceased (fig. 5, *B*). In the absence of frost no further action has developed in these particular structures. On the other hand, there is considerable evidence from structures located in higher altitudes to indicate that failures which may have been started by abnormal expansion have been greatly accelerated by freezing and thawing, ultimate disintegration resulting from a combination of the two.

There is evidence to indicate that some of the earlier failures on certain bridges in eastern Oregon (all of which have been repaired) were due to the use of unsound cement. If this is true, there is no particular mystery about these failures, as it is well known that so-called hard-burned free lime may exist in cement that has been improperly burned and that the gradual hydration of this lime within the hardened concrete will cause disruptive expansion. Manufacturers of portland cement claim that adoption of the autoclave test for soundness has entirely eliminated trouble from this source. The test was not in use at the time these bridges were built.

The most serious form of delayed expansion observed cannot be attributed to unsoundness in either cement or aggregates. In all three of the areas visited, rather conclusive evidence of "cement-alkali aggregate" reaction was observed. This type of cracking is generally assumed to be due to internal expansion resulting from a reaction between the alkalies (sodium and potassium oxides) in the cement with certain siliceous constituents of the aggregates. In certain cases opaline silica has been identified as the reactive material in the aggregate.

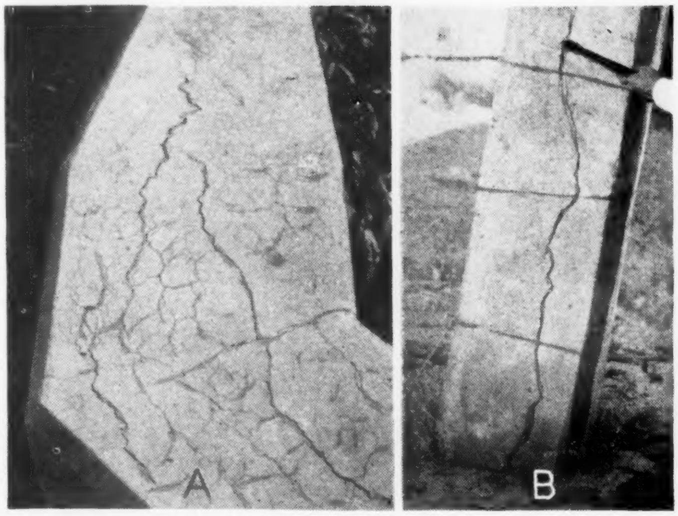


FIGURE 5.—EXAMPLES OF TYPE 4 DETERIORATION IN (*A*) NEBRASKA AND (*B*) CALIFORNIA IN WHICH THE MAIN PATTERN IS LONGITUDINAL.

In other cases certain igneous rocks, mostly volcanic, have been identified as reactive. So far, evidence of such a reaction has been based almost entirely on observation of structures in service and on expansion tests of specimens containing high- and low-alkali cements in combination with aggregates of various types. Tests and experience correlate very well insofar as the identification of combinations which cause trouble is concerned. However, the actual mechanism of the action is not yet fully understood although various agencies (notably the Bureau of Reclamation and the Portland Cement Association) are working on this phase of the problem.

Positive identification of the type or types of a particular failure is not always easy. This applies particularly to structures in which distress due to alternate freezing and thawing (type 2) may have been preceded by expansion due to an alkali-aggregate reaction (type 4). In such cases it is probable that the internal expansion due to chemical action may so weaken the concrete as to greatly lower its resistance to frost action. Some laboratory work recently reported by Tremper (7) indicates the possibility of such action. This theory also tends to explain the disturbing fact that some concrete structures built under modern specifications and under close engineering supervision, and which by all the rules should be durable, are not performing as satisfactorily as they should. However, the theory does not explain why many of the older structures located in the same areas are still in good condition. The cements used in the old structures (age 20 years or more) were probably as high in alkalies as were the corresponding cements made 10 or 12 years ago. The aggregates also were of the same general type. However, there were other differences in the cements which may explain the inability of some of the more recently constructed bridges to resist weathering as well as comparable structures built 20 or more years ago. This possibility will be discussed more fully later in the report.

During the course of the inspections, small hand samples of concrete were taken by the writer from a number of the bridges. These samples have been examined microscopically in the petrographic laboratory and the results reported in appendix I.

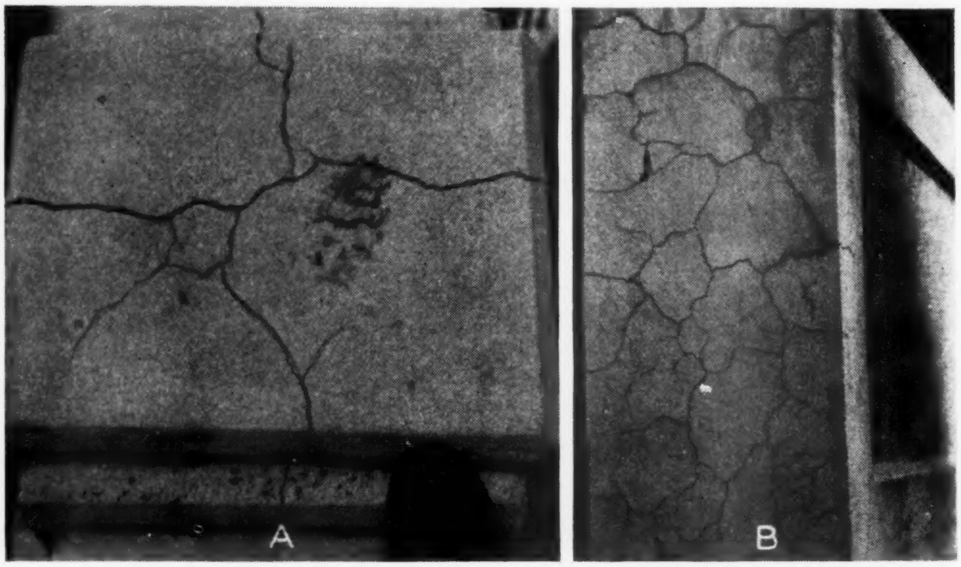


FIGURE 6.—EXAMPLES OF TYPE 4 DETERIORATION IN (*A*) NEBRASKA AND (*B*) WASHINGTON IN WHICH WIDE-OPEN RANDOM CRACKS DEVELOP.

FIGURE 7.—FAILURE OF A HANDRAIL IN WYOMING DUE TO ACCELERATED WEATHERING, AN EXAMPLE OF TYPE 2 DETERIORATION.

OBSERVATIONS IN WESTERN NEBRASKA, WYOMING, AND THE YELLOWSTONE AREA

During this inspection, some 40 or more structures located along the following route were observed. The party traveled from the Wyoming State line eastward on U S 30 to Kimball, Nebr.; on U S 30 westward from Cheyenne to Evanston, Wyo., thence north to the Jackson Hole area south of Yellowstone Park; thence through the park by way of Old Faithful, Madison Junction, Mammoth, Gardiner, Mont., northeast entrance road to Lamar River Bridge, the east loop road to Fishing Bridge; thence out of the park by the east entrance to Cody, Meeteetse, Thermopolis, Shoshoni, Casper, Douglas, Wheatland, and Cheyenne, Wyo.

Detailed discussion of the condition of the concrete in the structures inspected will not be attempted. It was impossible, in the limited time allowed, to make a detailed examination of all of the bridges or to take full and complete notes as to the condition of various portions of each structure. However, notes taken regarding the more important structures are in sufficient detail to give a very good idea of the condition of the concrete and related circumstances.

All four types of deterioration were observed on this particular trip. Several structures, particularly the older ones, showed no evidence of distress other than type 1. Among these may be mentioned an arch bridge over the Shoshone River just below the dam built in 1925; a bridge over the Big Horn River at Thermopolis built in 1922; a bridge over the North Platte at Casper built in 1921; a bridge over Deer Creek east of Glenrock built in 1921; and a bridge over the North Platte west of Douglas built in 1923. An old bridge over the Gibbon River, built by the War Department probably around 1913, was also in excellent condition considering its age. This bridge showed very little evidence of trouble except on one corner post, which was badly cracked. A sample from this post showed distinct evidence of an alkali-aggregate reaction (fig. 15, *A*).

Not all of the old bridges are in good condition, however. Mention should be made of two old bridges in the park built by the War Department, one over Can-

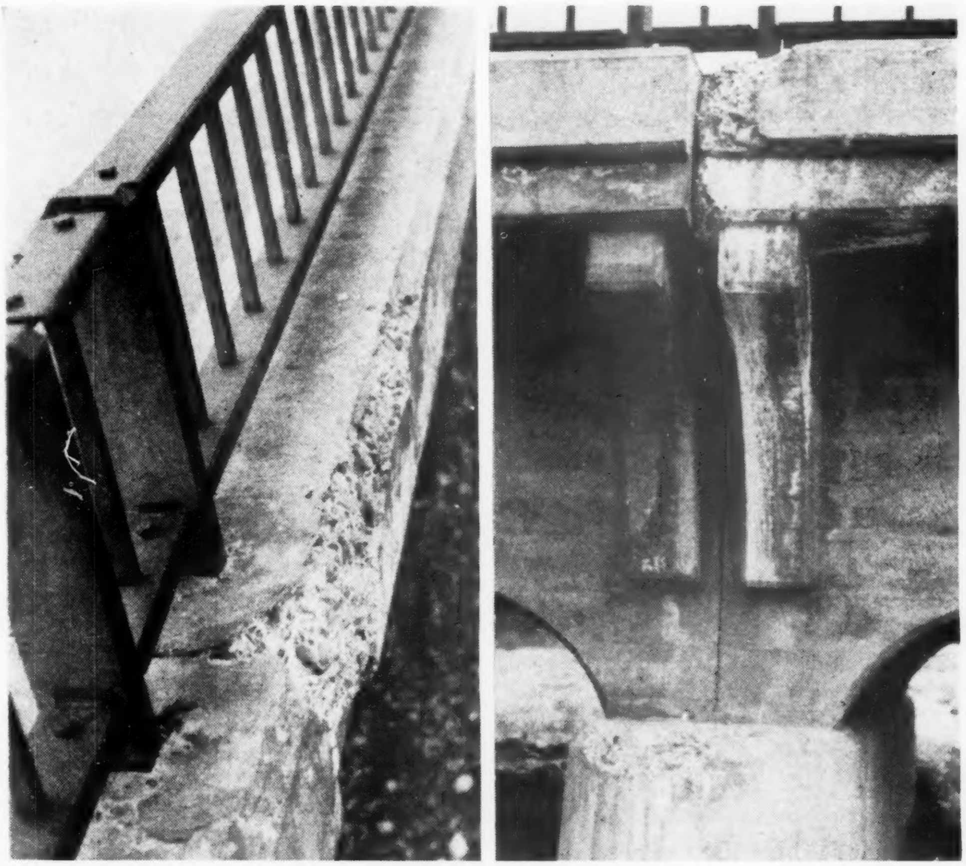


FIGURE 8.—AN EXAMPLE OF TYPE 2 DETERIORATION IN WYOMING DUE TO WEATHERING.

yon Creek and the other the Chittenden arch over the Yellowstone River. Both of these bridges show advanced disintegration due to weathering. There is no information as to when these bridges were built, but they are probably at least 30 years old.

Weathering (type 2) in all stages from initial D-line cracking to complete disintegration was observed in many structures both in and adjacent to the park as well as in other sections of Wyoming. Many examples were found which appeared to be simple cases of ordinary weathering. In other cases there was evidence that the effects of weathering may have been complicated by abnormal expansion of the type caused by an alkali-aggregate reaction.

Cases of simple type 2 weathering occurred in certain structures along U S 30 in Wyoming and in the superstructure of certain bridges in the Jackson Hole area, south of Yellowstone Park. In one of these structures, built in 1931, disintegration is confined almost entirely to the concrete handrails, end posts, and so forth. The apparent variability of the concrete, a rather typical condition, may be observed in figure 7, some of the rail being still in good condition. There is some evidence of abnormal expansion (other than might be caused by simple weathering) in the handrail of this structure. In this connection an investigation of the aggregates used in the bridge by the Bureau of Reclamation indicates that they are probably not alkali reactive. However, the bureau report does state that "certain of the weathered products could not be positively cleared of suspicion of susceptibility to alkali-aggregate reaction." It is proposed to investigate these materials further in the laboratory, including testing for expansion properties in mortar bars containing low- and high-alkali cements. Incidentally, reports indicate that, from the standpoint of physical soundness, the aggregates used in this structure were entirely adequate. The cement also was entirely satisfactory as measured by usual standards.

On another bridge built in 1931 and also located just south of the park, the distress appears to be entirely of the type 2 variety. There was no evidence of abnormal expansion due to chemical activity. In this structure there is considerable disintegration in the concrete brackets supporting the walk, in the curb supporting the steel handrail and in certain of the pier caps (fig. 8). Exposure conditions are extremely severe, temperatures ranging from 90° F. in the summer down to −50° F. in winter, with heavy precipitation and sufficient thawing in winter to produce a large number of alternations per year.

Within the park area proper, two bridges were of particular interest. One of these was built in 1939 and the other in 1940. Both bridges are showing signs of type 2 deterioration, evidenced by slight checking on the vertical surfaces of the curbs (fig. 2). The action is slight so far but is disturbing because the pattern is so familiar as the initial evidence of progressive disintegration.

Reference should also be made to a bridge over the Yellowstone River at Gardiner, Mont., built by the Public Roads Administration in 1930. This bridge is in excellent condition. Construction reports indicate that, with the exception of four cars, all of the sand was from a commercial pit located some distance from the project and that local sands were rejected due to low strength in mortar tests and an excess of obsidian.

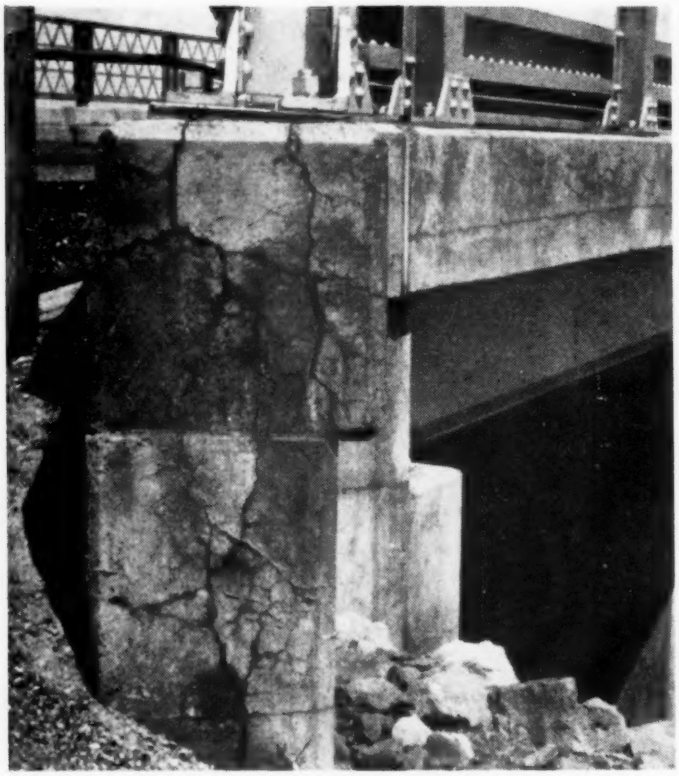


FIGURE 9.—PROBABLY AN EXAMPLE OF TYPE 4 DETERIORATION IN WYOMING SHOWING CRACKS IN THE END POST.

CAUSES OF DETERIORATION AND PREVENTIVE MEASURES DISCUSSED

The fact that some of the exposed concrete in the two park bridges as well as similar concrete in certain of the recently constructed bridges on the east approach road to the park is in the first stage of what appears to be type 2 weathering, is further evidence that even good concrete, according to usual standards, may be inadequate under this severe exposure. These bridges were built in accordance with the best engineering standards of the day, had adequate inspection, and certainly the high cement content used (6.5 sacks per cubic yard) would insure that water ratios did not exceed 6.0 gallons per sack. The concrete is not good enough and the question naturally arises—what can be done about it?

As to concrete in existing structures, it is strongly recommended that bridges showing this type of deterioration be waterproofed as soon as possible, using the linseed oil treatment which was proposed some years ago by the Portland Cement Association and successfully used by the Oregon State Highway Commission. In this connection, special attention is directed to the necessity for thoroughly repairing all affected areas by patching before the waterproofing is applied. This is the only way in which further progressive deterioration can be avoided. The methods used by the Oregon State Highway Commission for patching and waterproofing disintegrated concrete are outlined in appendix II.

For future work the writer recommends the use of an air-entraining agent in all exposed concrete in areas where severe frost action occurs. Considerations supporting this recommendation are discussed later.

In one of the Bureau of Reclamation reports relating to this problem, the opinion is expressed that the early failure of concrete in the bridges located south of the park may have been caused by the use of cements high in tricalcium aluminate (C_3A). The writer believes that the evidence that high C_3A content, per se, contributes to lack of durability is far from conclusive. Many of the older structures which are in excellent condition were made with cement high in C_3A. However, those older cements were not nearly so finely ground as modern cements. This point is also discussed later.

Evidence of type 3 deterioration (salt scaling) was confined to surfaces of pavements and sidewalks of structures where either sodium or calcium chloride had been used for ice removal. A particularly severe case was noted on a pedestrian underpass built in 1941. This trouble is so obviously due to salt treatment as to require little discussion except to call attention to the fact that air entrainment will very definitely increase resistance to this action. Resistance to salt action is highly important as the hazard from ice forming on concrete pavements is so great as to require the use of salts as a control measure even though there is danger of damaging the concrete. An air-entraining agent should be used in all concrete pavements which are apt to become hazardous through the accumulation of ice in winter.

Type 4 abnormal expansion was observed in pavements and grade-separation structures in western Nebraska, and in one structure on U S 30 in eastern Wyoming (fig. 9). Tests for expansion of mortar specimens containing the aggregates used in this bridge in combination with both high- and low-alkali cements are being made. The opinion has been expressed that the cracking in this structure may be due to differential expansion between the concrete and the steel handrail which is anchored in it, rather than to internal expansion of the concrete itself. This possibility must be examined.

Aside from the projects above noted and the old bridge in Yellowstone Park to which reference has been made, no other direct evidence of alkali-aggregate expansion was found during the inspection in District 3. However, as noted previously, the possibility of this reaction as a contributing factor in the disintegrating of these structures cannot be dismissed from consideration. It is quite possible, as Tremper's tests indicate, that the resistance of the concrete may have been weakened through the formation of fine expansion cracks caused by an alkali reaction. The intensity of the reaction in this area may have been considerably less than in the areas where it has been definitely identified, thus accounting for the absence of characteristic wide-open cracks. Aggregates in the Yellowstone Park area should be thoroughly examined with this possibility in mind.

OBSERVATIONS MADE IN OREGON AND WASHINGTON

During this portion of the trip approximately 120 bridges, and some pavements along the following route were examined.

In Oregon: From Ontario, north over U S 30 to Pendleton, thence south and west over U S 395 and U S 28 to Bend, thence south on U S 97 to Klamath Falls and returning to Bend; thence over the Cascades to Salem by way of the North Santiam River Highway; then to Portland.

In Washington: From Portland over U S 99 to Marys Corner; thence over No. 5 through the eastern portion of Mount Rainier National Park to Seattle; thence over U S 10 to Teanaway; thence through Blewett Pass to Peshastin; thence over No. 15 to the Berne undercrossing of the Great Northern Railroad,

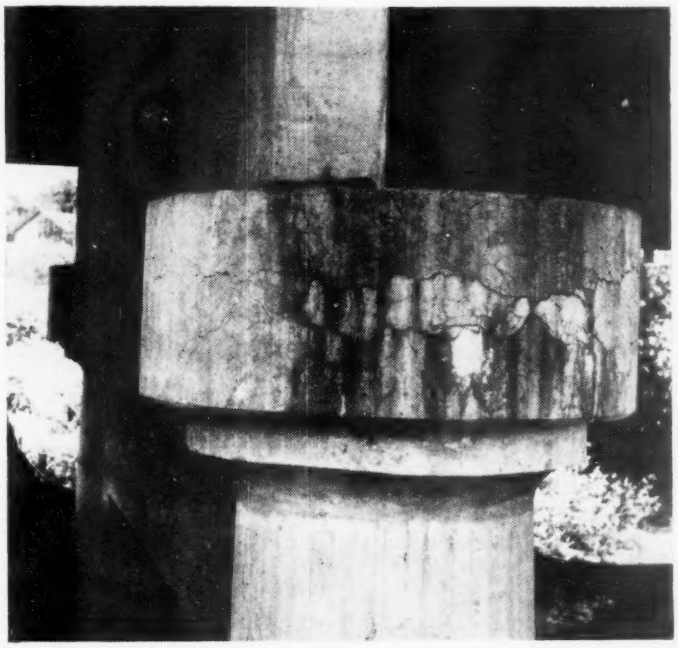


FIGURE 10.—POSSIBLY AN EXAMPLE OF TYPE 4 DETERIORATION IN OREGON SHOWING OPEN CRACKS IN THE PIER CAP.

thence to Wenatchee, thence easterly, following in general U S 10 alternate to Spokane; thence southwesterly through Ritzville, Lind, Pasco, Prosser, Goldendale, and along the Columbia River to Portland.

The routes followed in both States were chosen so as to include most of the areas in which trouble has been experienced. The fact that all of these areas lie either in or east of the Cascade Range, where climatic conditions are severe, and that no trouble has been experienced in the regions west of the mountains, indicates quite definitely that frost action is an important factor in the deterioration. The question is whether it is the only factor. There is evidence in both States to indicate that it may not be.

Most of the deterioration observed in eastern Oregon was of the familiar D-line or type 2 variety. Curbs and sidewalks of bridges were the most affected. Most of the precast rail pedestals were in good condition. Owing to the excellent bridge maintenance work of the Oregon Highway Department, most of the distress which caused so much concern in 1937 was not visible at the time of this inspection. This applies, among others, to several of the Burnt River bridges, to the Perry Arch over Grande Ronde River, to the overhead crossing at Meacham and to the Umatilla River bridge at Pendleton. The majority of the defects now showing are apparently the result of recent weathering on structures which have either been inadequately waterproofed or not waterproofed at all.

There is still evidence in places of the abnormal expansion which occurred in many of the bridges built with cement which may have been unsound. Some indications of continuing expansion were seen in one of the Burnt River bridges. The open cracks in the pier caps of another Burnt River bridge are indicative of abnormal expansion (fig. 10). As previously noted, an explanation of the excessive expansion of concrete containing a certain cement is that the cement in use at that time, due to improper manufacture, probably contained hard-burned free lime. However, according to State reports, this cement was also very high in

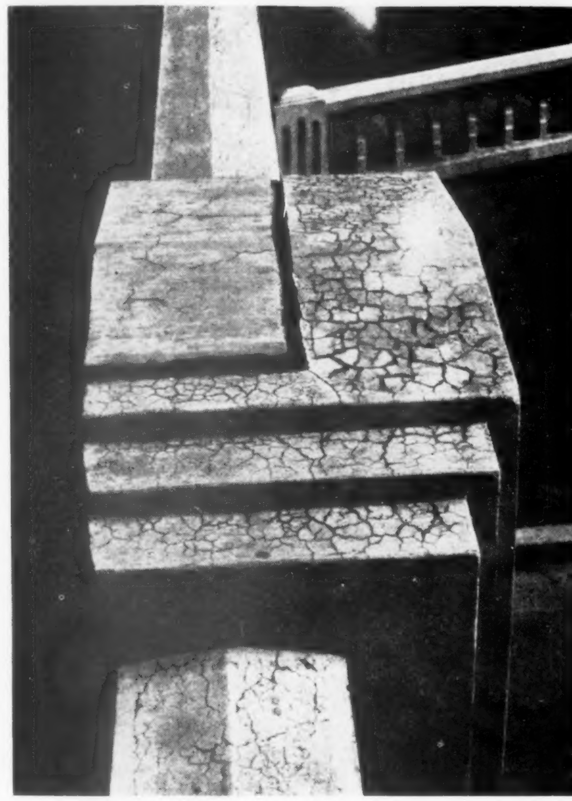


FIGURE 11.—POSSIBLY AN EXAMPLE OF TYPE 4 DETERIORATION IN OREGON SHOWING THE OPEN TYPE OF CRACKS.

alkalies, one report showing a value as high as 1.68 percent calculated as Na_2O. Therefore, the possibility of alkali-aggregate reaction cannot be definitely eliminated although it is understood that expansion bar tests run by the State on the aggregates used have not yet revealed unusual expansion. It is of interest to note that, of the 19 bridges reported to contain the high-alkali cement, 16 now show, or are reported to have shown, evidence of deterioration.

A view of wide pattern cracking, which may be type 4, is shown in figure 11. This is a grade-separation structure in central Oregon. The cracking seems to go beyond the familiar D-lines that are so prevalent on curbs and other thin sections.

Numerous structures were inspected in and around Klamath Falls. Many of these are in good condition, as for example, two bridges over the Link River, built in 1926 and 1931, and the Southern Pacific overcrossing north of Klamath Falls (1932). Others show extensive D-lines, principally in curbs and walks. This distress can probably be attributed to ordinary weathering. One of these bridges, built in 1930, shows evidence of expansion in longitudinal cracks in arch ribs. This bridge has been extensively repaired in an effort to prevent progressive disintegration. In spite of this there is some evidence of continuing disintegration in curbs and rails.

A recently constructed underpass built in 1939 is showing evidence of progressive disintegration. In this, as in many other structures, the disintegration is, in general, worse on the surfaces exposed to winter winds from the north. It is said that sleet frequently builds up on these surfaces while other surfaces are bare. This extreme exposure is particularly hard on concrete and it is quite possible that the disintegration

FIGURE 12.—AN EXAMPLE OF TYPE 4 DETERIORATION IN WASHINGTON SHOWING THE LONGITUDINAL TYPE OF OPEN CRACK.

in this area is entirely due to this cause. Here again the possibility of abnormal expansion due to chemical action within the concrete cannot be positively eliminated from the picture. Further investigations along this line must be made.

An interesting example of the possible effect of materials on the quality of concrete is found in the series of bridges along the North Santiam Highway, northwest of Bend. One of these, built in 1938, is showing serious disintegration, particularly on the railing. Sand and gravel from the Willamette River were used in this structure. On the other hand, several bridges along the same highway, built in 1934, in which local aggregates and a cement which has built up a very good service record, were used, are in very good condition. These materials are being studied in the co-operative investigation now under way.

MANY OLD STRUCTURES FOUND IN GOOD CONDITION

Many of the Oregon structures were in excellent condition. Among these were a number of old bridges on the old Oregon Trail, U S 30, between Ontario and Pendleton. These include Jacobson's Gulch Bridge, the overpass at Unity, an old bridge over the Grande Ronde River, and the overhead crossing at Glover. All four were built in 1922. The fact that so many of the old bridges are in good condition whereas many structures built much more recently are showing deterioration is further indication of the trend found on the inspection trip in District 3.

Considerably more of the wide-open type of cracking which is associated with abnormal expansion was observed in Washington than in Oregon. For example, in the area immediately adjacent to Mount Rainier National Park, disintegration of this type has been definitely identified as due to the use of aggregates from the Cowlitz and White Rivers in combination with cements high in alkalies. Bridges in the same general vicinity in which the same aggregates were used in combination with cements low in alkalies are in good condition. The condition of these bridges at the present time is about as described by Tremper in 1941, except that deterioration has progressed to some extent. In addition to deterioration in the superstructure, expansion cracks are now evident in the arch ribs and in various parts of the substructure of several of these bridges. Figure 12, showing longitudinal cracking in a rail coping, and figure 6, *B*, showing characteristic wide-open cracking in an end post, are typical.

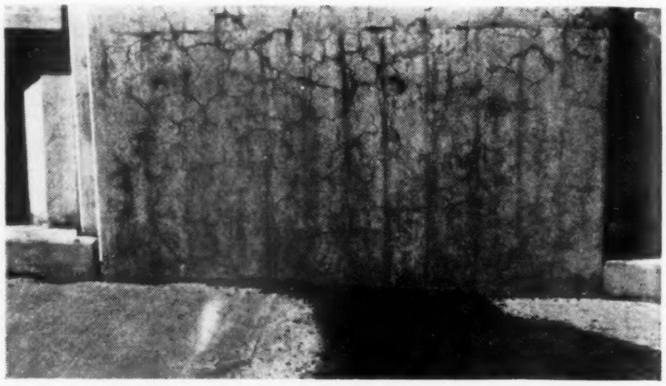


FIGURE 13.—PROBABLY EXAMPLES OF TYPES 2 AND 4 DETERIORATION IN WASHINGTON, SHOWING WELL-DEVELOPED PATTERN CRACKS.

Wide-open cracks indicative of expansion cracking were found in the northeast rail of a bridge on U S 99 over the Toutle River north of Kelso. This is the only bridge west of the mountains showing disintegration that came to the attention of the writer.

The inspections in central Washington included several bridges southwest and northwest of Wenatchee. Many of these showed characteristic evidence of type 2 weathering, that is D-lines on exposed members such as rails and curbs. Many of these structures also showed open cracks indicating the possibility of abnormal expansion. Aggregates which were used in a number of the bridges in this area have been selected for study in the cooperative work which has been started.

Typical type 2 D-line deterioration was found on a number of bridges in the vicinity of and to the southwest of Spokane. In some cases D-lines had progressed to the point of actual disintegration. As in other localities, the most seriously affected parts of the structures were the exposed sections above the deck line such as rails, curbs, and rail posts indicating that weathering is the primary cause of the trouble.

The concrete in one of the grade-separation structures in Spokane is of particular interest. Typical pattern cracking on one of the end posts is shown in figure 13. The cracks appear to go deeper than the fine D-line cracks so often noted. There was more of this type of cracking in Washington than in Oregon. The problem is whether this type of failure is purely the result of physical effects of weathering or whether, as Tremper's recent tests indicate, the resistance of the concrete has been weakened through some reaction involving the alkalies in the cement.

OBSERVATIONS IN CALIFORNIA

The inspections in California were made to observe the recent condition of structures in which alkali-aggregate reaction has taken place. These are located in the southern portion of the State. Structures in the high altitudes in which some type 2 weathering has been reported were not inspected. Several bridges over the Salinas River were examined and their present condition compared to that at the time of a previous inspection in 1937. Very little evidence of further action could be found, indicating that the chemical reaction which probably caused the initial expansion cracking has ceased. The cracking shown in figure 5, *B*, is typical of the vertical cracking noted on these structures. Other examples of this action may be seen in figure 14. In figure 14, *B*, the section of road surface back of the joint contains a reactive fine aggregate whereas in the section in the foreground the fine aggregate is nonreactive.

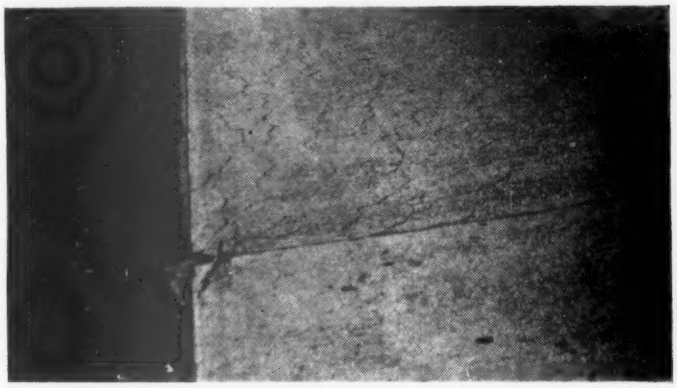
Deterioration resulting from cement alkali-aggregate reaction in California has been thoroughly discussed by Stanton and others and it is not necessary to go into detail in this report. However, attention should be called to the difference in conditions in southern California and in the northern States. In California, type 4 deterioration is not complicated by frost action. On the other hand, the concrete in the Mount Rainier area and in other sections of the north where there may be an alkali-aggregate reaction, is subject to many cycles of freezing and thawing each year. Cracks in the concrete due to expansion will obviously weaken the concrete and make it an easy prey to frost action. In this connection the combined effect of internal expansion and salt spray is seen in the serious disintegration of the Ventura sea wall, which is in very bad condition.

During the California inspection the party visited Parker Dam on the Colorado River and Copper Basin Dam, nearby. Both structures and also the pavement in the traffic circle at Fresno (*3*, *5*) show evidence of alkali-aggregate reaction. An interesting summary of the present knowledge of cement alkali-aggregate reaction, is contained in an appendix to the report of Committee C–1 on Cement, A. S. T. M., which appeared in 1943 (*1*). The summary contains a fairly complete list of references to reports on this subject.

WEATHERING AS AFFECTED BY DIFFERENTIAL WATER CONTENT

It is a basic principle of concrete making that, given sound materials, durability is governed largely by the quality of the cementing ingredient—that is, the cement paste—and, further, that the quality of the paste is controlled largely by its relative water content (the water-cement ratio). This principle is as firmly established today as it ever was although the relatively new principle of air entrainment may seem at first thought to run counter to the idea that durability is a function of the density of the cement paste.

The influence of the water-cement ratio on durability is so well recognized today that virtually all specifications for concrete to be exposed to severe weathering either require directly that the water-cement ratio not exceed 6.0 gallons per sack of cement, or specify requirements for minimum cement content and consistency in such a way as to insure that this value is not exceeded. A study of specifications governing construction of the various bridges inspected on this trip indicates that, in general, proportions were used which would insure compliance with this requirement in combining materials at the mixer. Unfortunately the process of handling and depositing concrete in the forms, even when done under good supervision, frequently results in segregation, nonuniform compaction (particularly at joints), bleeding or water gain, and so forth. Ideally, concrete should be absolutely uniform in composition but variable factors all combine to make a product which actually may be far from uniform. The result is that even though the water-cement ratio of the mix as designed may be within the required limits, the actual water content in certain parts of the structure may be too high for adequate protection. The nonuniform distribution of concrete within a member as well as batch-to-batch variations in the quality of the concrete as mixed, variations in curing conditions, etc., all undoubtedly combine to produce the nonuniformity in resistance to weathering which is so often observed in various parts of the same structures. It is felt that at least some of the troubles which were noted during these inspections can be attributed to these conditions.



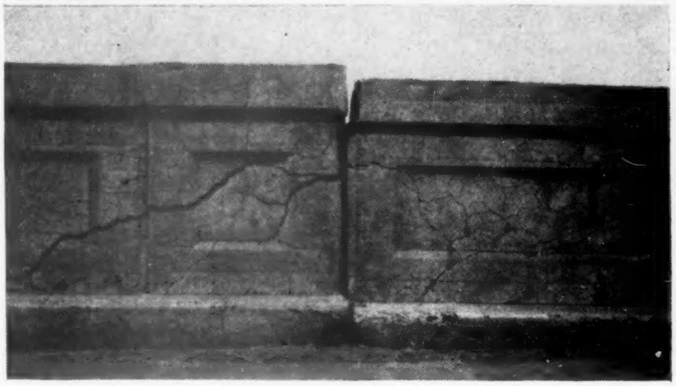


FIGURE 14.—TYPICAL TYPE 4 DETERIORATION CRACKING IN CALIFORNIA.

The writer would like to suggest adoption of a modification in placing procedure recently suggested to him by Mr. F. R. McMillan, of the Portland Cement Association. To prevent accumulation of a layer of relatively porous concrete of high water content in the top surface of vertical lifts, Mr. McMillan proposes the following: Instead of immediately striking off the concrete when the top of the form is reached, allow it to pile up 2 or 3 inches above the form, leaving it undisturbed for an hour or so in order that the excess water resulting from bleeding may accumulate in this layer. As soon as bleeding has ceased, strike off and finish in the usual way. The usual porous layer, instead of remaining in the concrete as a potential source

of weakness, will be wasted. The only loss will be a little concrete, the cost of which is a small price to pay for the elimination of a very common defect.

AIR ENTRAINMENT DISCUSSED

Application of the principle of air entrainment offers the most promising immediate solution to the problem of how to obtain more durable concrete. Originally suggested as a means for controlling salt scaling on pavements, it now appears that, in addition, resistance to freezing and thawing in general is greatly improved by the use of an air-entraining agent in the concrete. The effectiveness of air entrainment seems to lie in the fact that numerous minute air voids are incorporated in the concrete, and that these voids seem to act as cushions against the expansive force produced by freezing water. In any event, both laboratory tests and field experience indicate that substantial improvement in durability is obtained with little sacrifice in strength provided proper control is exercised in the use of the material.

At the present time air can be entrained in two ways; first, by the use of "Air-Entraining Portland Cement," a cement in which the air-entraining agent has been interground during manufacture and second, by adding the air-entraining agent directly to the concrete at the time of mixing. For the time being the writer favors the practice of adding the material at the mixer for several reasons. The amount of air that will be entrained in any particular case depends upon a number of factors in addition to the amount of active agent in the cement. The kind and grading of the aggregates, the type of mixer and the mixing time all affect the result, so that merely fixing the amount of air-entraining agent to be carried by the cement, or even fixing the amount of air which will be entrained in a standard test mortar as is done in the latest revision of A. S. T. M Specification C–175 is not sufficient. The only way in which the effect of all of the variables can be taken into account is to place control in the hands of the engineer on the job, permitting him to adjust the amount of air-entraining agent depending upon the particular conditions under which he is working.

It is quite important that close control be exercised because of the rather narrow limits of air content that must be maintained for optimum results. At the present time it seems that a total air content of from 3 to 5 percent based on the theoretical weight of the air-free concrete will be about right. Less than 3 percent may not be sufficient for maximum durability, whereas when the total air content exceeds 5 percent a substantial loss in strength without any further increase in durability may result. In a report published in June 1944 H. F. Gonnerman (*2*) of the Portland Cement Association presents the results of tests showing the effect of using both air-entraining portland cements and air-entraining materials added to the batch at the mixer. Several air-entraining agents were tested and if properly used should give satisfactory results. However, at the moment, the material supported by the greatest background of actual experience is Vinsol resin. This material has been used to a considerable extent interground with the cement. It has also been used, in the form of a Vinsol resin-sodium hydroxide solution, by adding to the batch at time of mixing. A weight test on the fresh concrete will indicate quite accurately the amount of this solution to add in any given case to produce the required air content. In addition to Vinsol resin, the material known commercially as Darex has also been approved for use in the manufacture of air-entraining portland cement, under A. S. T. M. Specification C–175. This material may also be added at the mixer as an air-entraining agent.

Although the writer favors the practice of adding the air-entraining agent at the mixer, it is realized that there are certain construction problems involved in this practice which are avoided by the use of air-entraining cement. The difficulty of properly controlling the addition on the job of very small quantities of such active materials as Vinsol resin is recognized. With Vinsol resin there is also the problem of preparing the material for use. To many engineers these practical matters outweigh the theoretical advantages of the procedure recommended in this report. However, it should be emphasized again that the really important matter is the amount of air which is entrained in the concrete as mixed on the job.

Not only must the air content be sufficiently high to insure the desired improvement in durability but it must not be so high as to jeopardize strength, bond-with-steel or any other essential property of the concrete. The optimum range appears to correspond to a total air content of from 3 to 5 percent. This is a rather narrow range and will require close control. Regardless of the method used to obtain air entrainment (by addition of an air-entraining agent at the mixer or the use of air-entraining cement), provision for such control during construction must be made in the specifications which govern the work. This can be done through the use in the field of a weight test such as A. S. T. M. C—138–44, with provision in the specifications for making such changes in materials, proportions, or methods of mixing as may be necessary to obtain the proper air content at all times.

There are certain indirect benefits associated with the use of air-entraining agents that should be mentioned. They have a distinctly plasticizing effect, especially in the leaner mixes, and, in general, reduce bleeding or water gain substantially. On this account it should be possible to obtain more uniform placement and thus avoid some of the segregation troubles to which reference has already been made. The additional plasticity makes it possible to reduce the sand content about 3 percent, thus at least partially compensating for the increased yield (lower cement content) which results from the air entrainment. This adjustment also tends to overcome the loss in strength which almost invariably accompanies increase in air content.

MODERN VERSUS OLD-FASHIONED CEMENT

Many of the older bridges covered by these inspections were in surprisingly good condition. In reviewing their condition with respect to age they seem to fall roughly into two groups; those built during the twenties and earlier and those built subsequent to 1930. It can be stated definitely that, on the whole, the bridges in the first group show less disintegration than those in the second group. Of 36 bridges built before 1930—24, or 67 percent, were free from defects other than those which could be classified as type 1 deterioration. On the other hand, of 101 bridges built after that date only 28, or 27 percent, were so classified. A similar comparison has been furnished by Mr. Paxson. He lists 61 structures on U S 30, between Umatilla and Ontario. Of the 37 built prior to 1930—29, or 78 percent, have

according to his record shown no evidence of weathering, whereas of the 24 built after that date only 9, or 37 percent, are so listed.

It is reasonable to suppose that, by and large, construction procedures are just as good now as they were 20 years ago. Presumably they are better. Likewise, engineering supervision has, on the average, improved. Certainly it has not deteriorated. The general character of the aggregates has not changed, although methods of processing, grading, and other factors tending to uniformity have improved. In view of these facts, it is not surprising that engineers should wonder about the cement. It is the one remaining factor. Is there some characteristic of modern cements not found in the old-fashioned type that tends to lower resistance to weathering?

The one outstanding difference is, of course, fineness. In 1917 the A. S. T. M. specifications required that not more than 22 percent be retained on the No. 200 sieve. The current specifications for type I portland cement requires a minimum specific surface of 1,600 square centimeters per gram, a limit which is roughly equivalent to 3 or 4 percent retained on the No. 200 sieve. The increase in fineness has come about gradually and largely as the result of demands on the part of users for high early strength. This demand culminated in 1930 in the adoption of specifications for high early strength cement, with still greater fineness.

Quite recently a very interesting theory has been advanced which may account for the apparent fact that modern cements, ground to a fineness corresponding to a specific surface of, say 1,800 square centimeters per gram do not make as durable concrete as the more coarsely ground cements in use 25 years ago. According to this theory, the maximum limit of 2 percent sulfur trioxide which has been carried for years in the A. S. T. M. specifications, while satisfactory for the coarser ground product, is not high enough for the more active, finer ground, modern cements. This applies particularly to cements high in tricalcium aluminate (C_3A) as it is largely for the purpose of regulating the hydration of this particular compound that the gypsum is employed. Recent investigations by the Portland Cement Association indicate that from the standpoint of both strength and drying shrinkage, cements of high and moderately high C_3A are improved by adding higher percentages of gypsum than are allowed by the present specifications. Furthermore, even cements of low C_3A content appear to be similarly improved if the alkali content is high. There are also some indications that drying shrinkage may be influenced by the alkali content of the cement independently of the C_3A content. The association is now engaged in an intensive study of this problem, with the idea of determining by means of laboratory freezing and thawing tests whether the use of larger quantities of gypsum will improve durability. If this is found to be the case, the sooner the specifications are changed the better.

NEED FOUND FOR RESTRICTION OF ALKALI CONTENT OF CEMENT FOR USE IN CERTAIN AREAS

This inspection has confirmed the belief that there is need for restrictions on the alkali content of portland cement when used in areas where alkali-reactive aggregates are found. It is true that all of the aggregates in these areas, may not be reactive. However, at the present time there are no tests, so far as the writer knows, that will reveal this characteristic in a reasonable length of time. The conventional expansion bar test requires several months at least. Furthermore, as pointed out by Tremper, cements high in alkali may react with certain aggregates in such a way as to weaken the resistance of the concrete to freezing and thawing even though the reaction is not sufficient to cause expansion in the bar test. For these reasons it seems wise, at least for the time being, to restrict the percentage of sodium and potassium oxide in the cement to 0.60 or less, calculated as Na_2O, in all areas where there is any evidence that reactive aggregates may be found.

This survey indicates that preventive steps should be taken in the area in southern California to which attention has been called by Stanton and others; in eastern Oregon and eastern Washington, including the area adjacent to Mount Rainier National Park; in the Yellowstone National Park, and possibly other areas in Wyoming; and in the area around Kimball, Nebr. Further information regarding the cement alkali-aggregate reaction will be available at the conclusion of the cooperative tests now under way.

SUMMARY OF RECOMMENDATIONS

For convenience the recommendations made in this report, insofar as they have application in the immediate revision of specifications, are summarized below:

That, where concrete is to be exposed to severe frost action, specifications be revised, where necessary, to require, in addition to a suitable cement content, that the free water content of the mix shall, in no case, exceed 6.0 gallons per sack of cement.

That, where concrete structures or portions of structures (including pavements) will be exposed to severe frost action, provision be made to entrain sufficient air in the fresh concrete so as to produce a total air content of from 3 to 5 percent, based on the theoretical weight of the air-free concrete.

That the desired air entrainment be obtained preferably by adding an approved air-entraining agent to the concrete at the time of mixing, in such quantity as will maintain the percentage of air within the limits specified.

That approval of any material proposed for use as an air-entraining agent be based on data obtained from either research or field use, or both, that are sufficiently comprehensive to demonstrate to the satisfaction of the contracting agency that the proposed material, when used as required, will not seriously affect the strength or other essential properties of the concrete.

That, when the desired air entrainment is to be obtained by the use of an air-entraining cement: (1) The cement meet the requirements for air entrainment given in A. S. T. M. Specification C-175-44 T; and (2) the specifications authorize the engineer to require such changes in materials, proportions, or methods of mixing as may be necessary from time to time to maintain the percentage of air within the limits specified.

That, in all areas where tests or previous experience indicate that alkali-reactive aggregates may be encountered, specifications for portland cement be modified by requiring that the total percentage of sodium oxide plus 0.658 times the percentage of potassium oxide shall not exceed 0.60.

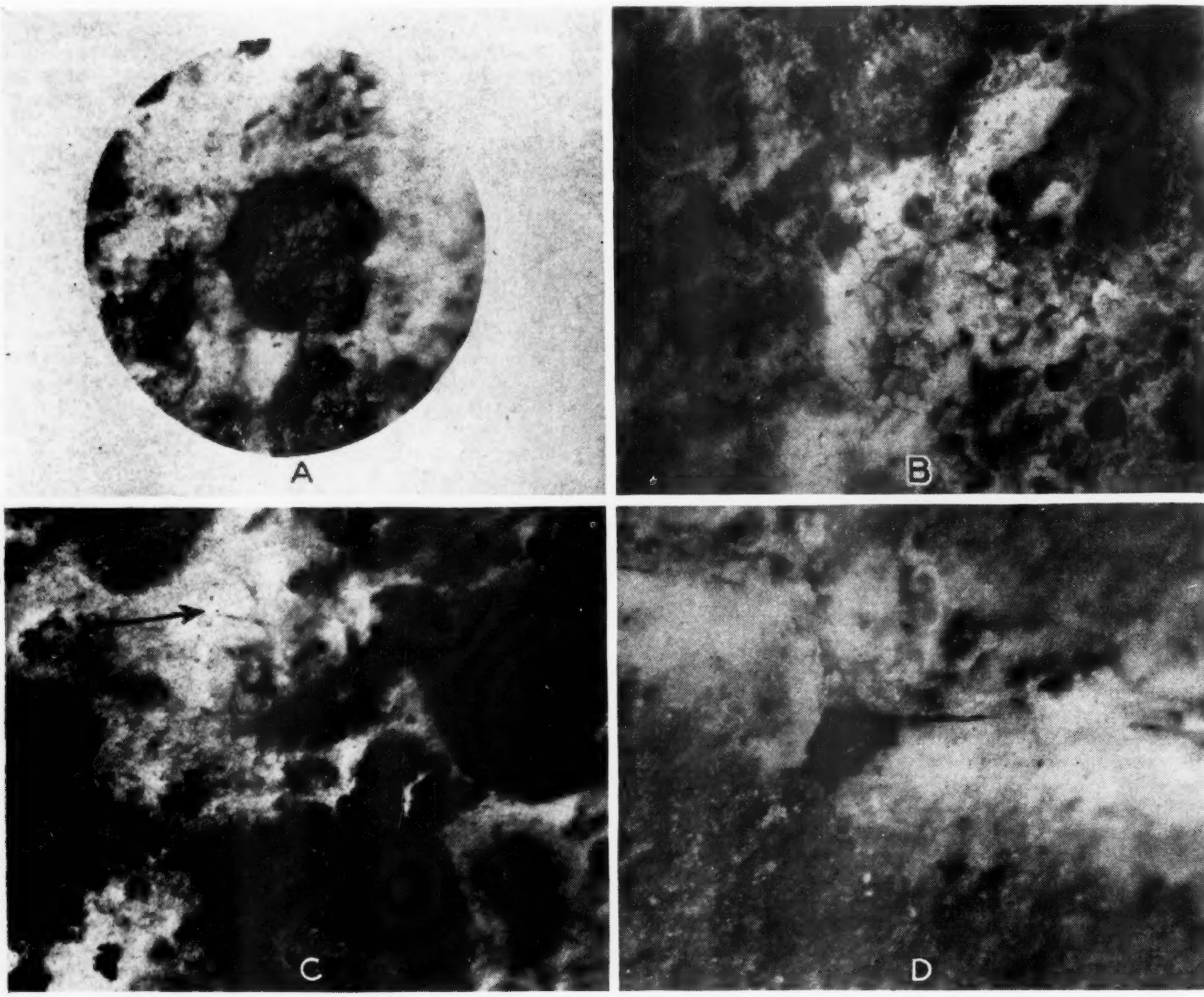


FIGURE 15.—(*A*) DEPOSIT OF GEL SHOWING SHRINKAGE CRACKS ON CLEAR FORK CREEK BRIDGE IN WASHINGTON, (*B*) THE FRACTURED SURFACE OF AUTOCLAVE BAR SHOWING A DEPOSIT OF GEL, (*C*) DEPOSIT OF GEL SHOWING SHRINKAGE CRACKS ON GIBBON RIVER BRIDGE IN YELLOWSTONE NATIONAL PARK, AND (*D*) GEL-FILLED CRACK ON SURFACE OF CONCRETE ON DEADWOOD CREEK BRIDGE IN WASHINGTON.

APPENDIX I.—REPORT OF PETROGRAPHIC LABORATORY ON SAMPLES OF CONCRETE OBTAINED DURING INSPECTION TRIP

During these inspections, a number of small samples of concrete were collected. In some instances samples were taken from portions of structures in which distress was very evident, while in other cases the samples were collected from apparently sound concrete. These samples were from the following bridges:

Gibbon River Bridge, Yellowstone Park.
North Santiam River Bridge, Oregon.
Marion Creek Bridge, Oregon.
Purcell Creek Bridge, Washington.
Clear Fork Creek Bridge, Washington.
Deadwood Creek Bridge, Washington.
Bridge in Tumwater Canyon, Washington.
Monterey Trestle, California.

Samples from the Marion Creek bridge in Oregon and the Purcell Creek bridge in Washington appeared sound in every respect. The concrete was homogeneous, dense, and as far as could be observed there were no indications of alkali reaction.

The sample of concrete from the Gibbon River bridge in Yellowstone National Park had an excessive number of voids throughout. Several pockets of dried gel-like material were observed. Calcium-sulphoaluminate crystals were also noted in some of the voids and aggregate sockets. A few of the aggregate particles exhibited rims of weathering which may or may not have been pre-concrete in origin. Figure 15, *A* shows a gel deposit in one of the voids.

Remains of gel formation, showing characteristic shrinkage cracks, were observed in the concrete taken from the Clear Fork Creek and Deadwood Creek bridges in Washington. The sample from the Clear Fork Creek bridge exhibited indications of alkali-aggregate reaction in the formation of a partially transparent exudation on some of the aggregate faces as well as in the voids. Figure 15, *B*, illustrates a deposit of gel formation in concrete from this structure.

Aside from the gel formations in voids and on aggregate faces, the concrete from the Deadwood Creek bridge contained dried gel in one of the radiating cracks on one face of the concrete that had been exposed to the elements. This gel had caused a slight expansion in the concrete adjoining the crack. A view of this is shown in figure 15, *C*.

The sample taken from the bridge in Tumwater Canyon, Washington, was characterized by lack of gel relics, but an excessive amount of calcium-sulphoaluminate needles was noted. Considerable carbonation was also observed in some of the aggregate sockets. These conditions indicate the possibility of freezing and thawing action as the reason for distress noted in the concrete.

Deposits of a white, semihard substance were observed in the concrete taken from the North Santiam River Bridge, Oregon. These deposits were found not only in the aggregate sockets but scattered throughout the matrix as well. This material proved

to be a carbonate and a gel-like substance. Considerable quantities of calcium-sulphoaluminate needles were also observed. These observations point strongly to distress caused by the action of freezing and thawing.

The concrete from the Monterey Trestle, California, although apparently sound in appearance, contained a few areas of opaline material. Some slight evidence of alkali-aggregate reaction was observed in a few areas showing the characteristic gel shrinkage cracks.

Figure 15, *D* illustrates a gel deposit found in a broken autoclave bar fabricated with a fine aggregate containing opal.

APPENDIX II.—METHODS USED BY THE OREGON STATE HIGHWAY DEPARTMENT FOR PATCHING AND WATERPROOFING DISINTEGRATED CONCRETE

The following is an outline of the procedure used by the Oregon State Highway Department in patching disintegrated concrete and waterproofing concrete surfaces for the prevention of disintegration. Considerable success has been attained in Oregon by following the methods as herein described.

REMOVAL AND PATCHING OF DISINTEGRATED CONCRETE

When disintegration of the commonly called "D-line" type has taken place, it usually affects the edges and corners of curbs handrails, wingwalls, etc. To prevent further progression of this type of disintegration, it is necessary to completely remove all disintegrated material, great care being taken to reach sound concrete beyond the extent of the "decayed" area. This can be accomplished by the use of hammers and chisels on small areas or by paving breakers or chipping hammers on larger areas. If the disintegration has reached an advanced stage, the complete removal and replacement of that portion of the structure may be necessary. The importance of the removal of all traces of disintegrated material cannot be overemphasized. Experience has shown that often the workman will remove all visible affected material, then place a patch on what was, in his judgment, sound concrete only later to discover that the material adjacent to the patch continues to disintegrate. A good rule seems to be to remove all material thought necessary, and then remove some more.

After the removal of the disintegrated concrete a patch is applied, the success of which depends upon, (*a*) securing a bond to the parent concrete, (*b*) overcoming the tendency of the patch to shrink after placement, and (*c*) proper curing. All places to be patched should be chipped out to secure not less than three-fourths-inch thickness for the patch. The edges of the patch should be square, that is, no feather edges. All surfaces should be clean and rough so as to secure a good bond and should be thoroughly saturated by several applications of water. The preshrinkage of mortar is required for all patches. This is done by mixing the mortar well ahead of use and letting it stand. The time required for preshrinkage of mortar varies with the different brands of cement and climatic conditions of temperature and humidity, and is best determined by experiment on the job. In general, the mortar thus preshrunk should be susceptible to use without the addition of more water before reworking it for application. After the patch has been applied, proper care in curing must be taken by keeping the patch covered with wet burlap 6 to 10 hours, after which it can be covered with damp sand or burlap until the concrete has thoroughly taken its set.

New concrete patches should be allowed one or two weeks to dry out before applying the waterproofing treatment. New concrete should be given a neutralizing wash to prevent saponification of the linseed oil used in the waterproofing treatment. A solution consisting of three pounds of zinc-sulphate crystals to a gallon of water is brushed over the surface to be treated and allowed to dry to 48 hours. When thoroughly dry, all crystals on the surface are removed by wire brushing. This treatment is not necessary on old concrete.

WATERPROOFING TREATMENT

Cleaning.—Before the waterproofing treatment is applied, it is necessary that the concrete surface be clean and dry. To clean the surface, brush with a wire bristle brush removing all dust and loose material. Road oil or grease can be removed by scrubbing with gasoline or a solvent. Efflorescence can be removed by scrubbing with a 10-percent solution of hydrochloric acid. When water is used in cleaning, ample time must be allowed to permit thorough drying of the concrete surfaces before applying the waterproofing.

Application of Linseed Oil.—After the surface has been properly prepared and is clean and dry, two coats of hot linseed oil are to be applied.

The first coat consists of a mixture of 50 percent raw linseed oil and 50 percent turpentine heated to a temperature of 175° F. and applied with an ordinary paint brush. Better results will be obtained if atmospheric temperature is above 65°. Allow the first coat to set 24 hours before applying second coat.

After the first coat of the linseed oil-turpentine mixture has set, spots will be evident where the concrete is more porous than the remainder of the surface treated. These spots should be spot treated with the hot mixture and allowed to set before the second coat of linseed oil is applied.

The second coat shall consist of undiluted raw linseed oil heated to 175° and applied in the same manner as the first coat. When this coat is thoroughly dry, the surface can be painted with an approved concrete paint.

CONCRETE PAINT

After the waterproofing treatment has been applied, the entire treated surface should be given two coats of any standard outside white lead and oil paint tinted to the desired shade. A concrete color can be obtained by the addition of lampblack and raw sienna ground in oil. The Oregon standard white paint formula is as follows:

Paint composition:	*Percent*
Pigment not less than	70
Vehicle not more than	30
Pigment composition:	
White lead carbonate	40.0 to 45.0
Titanium barium pigment	35.0 to 40.0
Zinc oxide	15.0 to 20.0
Tinting pigments, if required	0.0 to 5.0

The first coat should be thinned by the addition of 2 quarts of turpentine and 2 quarts of boiled linseed oil to the gallon of the above formula. The second coat should be thinned with about 1 quart of boiled linseed oil to the gallon of paint so as not to get a heavy pigment coat that will be susceptible to scaling, but which must be heavy enough to brush out uniformly and evenly.

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THE FEDERAL-AID HIGHWAY ACT OF 1944

[PUBLIC LAW 521—78TH CONGRESS]

AN ACT

To amend and supplement the Federal-Aid Road Act, approved July 11, 1916 as amended and supplemented, to authorize appropriations for the post-war construction of highways and bridges, to eliminate hazards at railroad-grade crossings, to provide for the immediate preparation of plans, and for other purposes.

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That, when used in this Act, unless the context indicates otherwise—

The term "construction" means the supervising, inspecting, actual building, and all expenses incidental to the construction or reconstruction of a highway, including locating, surveying, and mapping, costs of rights-of-way, and elimination of hazards of railway-grade crossings.

The term "urban area" means an area including and adjacent to a municipality or other urban place, of five thousand or more, the population of such included municipality or other urban place to be determined by the latest available Federal census. The boundaries of urban areas, as defined herein, will be fixed by the State highway department of each State subject to the approval of the Public Roads Administration.

The term "rural areas" means all areas of the State not included in "urban areas".

The term "secondary and feeder roads" means roads in rural areas, including farm-to-market roads, rural-mail routes, and school-bus routes, and not on the Federal-aid system.

SEC. 2. For the purpose of carrying out the provisions of the Federal Highway Act, approved November, 9, 1921, as amended and supplemented, there is hereby authorized to be appropriated the sum of $1,500,000,000 to become available at the rate of $500,000,000 a year for each of three successive post-war fiscal years: *Provided,* That of the sums authorized to be appropriated for the first of such fiscal years $100,000,000 may be appropriated in accordance with the provisions of this Act to become available immediately upon apportionment of the authorization for said fiscal year for the making of surveys and plans and for construction: *Provided further,* That except for the sum appropriated pursuant to the preceding proviso, no part of the funds made available pursuant to this Act shall be used to pay costs incurred under any construction contract entered into by any State before the beginning of the first post-war fiscal year. The first post-war fiscal year shall be that fiscal year which ends on June 30th following the date proclaimed by the President as the termination of the existing war emergency, or following the date specified in a concurrent resolution of the two Houses of Congress as the date of such termination, or following the date on which the Congress by a concurrent resolution of the two Houses finds as a fact that the war emergency hereinbefore referred to has been relieved to an extent that will justify proceeding with the highway construction program provided for by this Act, whichever date is the earliest. The authorization for the first post-war fiscal year shall be apportioned among the States within thirty days from the passage of this Act. The authorization for the second post-war fiscal year shall be apportioned among the States within twelve months after the date of such termination or finding as above specified, and the authorization for the third post-war fiscal year shall be apportioned among the States within twelve months after the apportionment of the authorization for the second post-war fiscal year. As soon as the funds for each of the post-war fiscal years have been apportioned, the Commissioner of Public Roads is authorized to enter into agreements with the State highway departments for the making of surveys and plans, the acquisition of rights-of-way, and the post-war construction of projects. His approval of any such agreement shall be a contractual obligation of the Federal Government for the payment of its pro rata share of the cost of construction: *Provided, however,* That the Commissioner of Public Roads shall not, as a condition of approval of any project for Federal aid hereunder, require any State to acquire title to, or control of, any marginal land along the proposed highway in addition to that reasonably necessary for road surfaces, median strips, gutters, ditches, and side slopes and sufficient width to provide service roads for adjacent property to permit safe access at controlled locations in order to expedite traffic, promote safety, and minimize roadside parking.

SEC. 3. The sum authorized in section 2 for each year shall be available for expenditures as follows:

(a) $225,000,000 for projects on the Federal-aid highway system.

(b) $150,000,000 for projects on the principal secondary and feeder roads, including farm-to-market roads, rural free delivery mail and public-school bus routes, either outside of municipalities or inside of municipalities of less than five thousand population: *Provided,* That these funds shall be expended on a system of such roads selected by the State highway departments in cooperation with the county supervisors, county commissioners, or other appropriate local road officials and the Commissioner of Public Roads: *Provided further,* That in any State having a population density of more than two hundred per square mile, as shown by the latest available Federal census, the said system may be selected by the State highway department with the approval of the Commissioner of Public Roads without regard to included municipal boundaries: *Provided further,* That any of such funds for secondary and feeder roads which are apportioned to a State in which all public roads and highways are under the control and supervision of the State highway department may, if the State highway department and the Commissioner of Public Roads jointly agree that such funds are not needed for secondary and feeder roads, be expended for projects in such State on the Federal-aid highway system.

(c) $125,000,000 for projects on the Federal-aid highway system in urban areas.

SEC. 4. After making the deductions for administration, research, and investigations as provided in section 21 of the Federal Highway Act of 1921, the sums authorized shall be apportioned as follows:

(a) The $225,000,000 per year available for projects on the Federal-aid highway system shall be apportioned among the States as provided in section 21 of the Federal Highway Act.

(b) The $150,000,000 per year available for projects on the secondary and feeder roads shall be apportioned among the States in the following manner: One-third in the ratio which the area of each State bears to the total area of all the States; one-third in the ratio which

the rural population of each State bears to the total rural population of all the States, as shown by the Federal census of 1940; and one-third in the ratio which the mileage of rural delivery and star routes in each State bears to the total mileage of rural delivery and star routes in all the States: *Provided*, That no State shall receive less than one-half of one per centum of each year's allotment under subsection (a) and this subsection.

(c) The $125,000,000 per year available for projects on highways in urban areas shall be apportioned among the States in the ratio which the population in municipalities and other urban places, of five thousand or more, in each State bears to the total population in municipalities and other urban places, of five thousand or more, in all the States as shown by the latest available Federal census: *Provided*, That Connecticut and Vermont towns shall be considered municipalities regardless of their incorporated status.

(d) Any sums apportioned to any State under the provisions of this section shall be available for expenditure in that State for one year after the close of the fiscal year for which such sums are authorized, and any amount so apportioned remaining unexpended at the end of such period shall lapse: *Provided*, That such funds shall be deemed to have been expended if covered by formal agreement with the Commissioner of Public Roads for the improvement of a specific project as provided by this Act.

SEC. 5. (a) The Federal share payable on account of any project provided for by the funds made available under the foregoing provisions of this Act shall not exceed 50 per centum of the construction cost thereof other than costs of rights-of-way, and as to costs of rights-of-way shall not exceed one-third of such costs: *Provided*, That in the case of any State containing unappropriated and unreserved public lands and nontaxable Indian lands, individual and tribal, exceeding 5 per centum of the total area of all lands therein the Federal share shall be increased in each of the three post-war years by a percentage of the remaining cost equal to the percentage that the area of all such lands in such State is of its total area: *Provided further*, That the entire construction cost of projects for the elimination of hazards of railway-highway crossings, including the separation or protection of grades at crossings, the reconstruction of existing railroad grade crossing structures, and the relocation of highways to eliminate grade crossings, may be paid from Federal funds, except that not more than 50 per centum of the right-of-way and property damage costs, paid from public funds, on any such project, may be paid from Federal funds: *Provided further*, That not more than 10 per centum of the sums apportioned to any State under the terms of this Act for each of such post-war fiscal years shall be used for such railway-highway projects, to be expended in accordance with the Federal Highway Act, as amended and supplemented, and the provisions of this section.

(b) Any railway involved in any project for the elimination of hazards of railway-highway crossings paid for in whole or in part from funds made available under this Act, shall be liable to the United States for a sum bearing the same ratio to the net benefit received by such railway from such project that the Federal funds expended on such project bear to the total cost of such project. For the purposes of this subsection, the net benefit received by a railway from any such project shall be deemed to be the amount by which the reasonable value of the total benefits received by it from such project exceeds the amount paid by it (including the reasonable value of any property rights contributed by it) toward the cost of such project; and in no case shall the total benefits to any railway or railways be deemed to have a reasonable value in excess of 10 per centum of the cost of any such project. The liability of any railway to the United States with respect to any such project may be discharged by paying to the United States, within six months after the completion of such project, such amount as the Commissioner of Public Roads determines to be the amount of such liability. Any such determination of the Commissioner shall be made on the basis of recommendations made to him by the State highway department and on the basis of such other information and investigation, if any, as the Commissioner deems necessary or proper. If any such railway has failed so to discharge its liability to the United States with respect to any project within six months after the completion thereof, the Commissioner of Public Roads shall request the Attorney General to institute proceedings against such railroad for the recovery of the amount for which it is liable under this subsection. The Attorney General is authorized to bring such proceedings on behalf of the United States in the appropriate district court of the United States, and the United States shall be entitled in such proceedings to recover such sums as it is considered and adjudged by the court that such railway is liable for in the premises. Any amounts paid to or recovered by the United States under this subsection shall be covered into the Treasury as miscellaneous receipts.

SEC. 6. If the Commissioner of Public Roads shall determine that it is necessary for the expeditious completion of projects undertaken pursuant to this Act, he may advance to any State from funds heretofore or hereafter made available the Federal share of the cost thereof to enable the State highway department to make prompt payments for work as it progresses: *Provided*, That such State, after June 30, 1945, does not divert to other than highway uses road user revenues in violation of section 12 of the Highway Act of June 18, 1934. The funds so advanced shall be deposited in a special trust account by the State treasurer, or other State official authorized under the laws of the State to receive Federal-aid highway funds, to be disbursed solely upon vouchers approved by the State highway department for work actually performed in accordance with plans, specifications, and estimates approved by the Public Roads Administration under the provisions of this Act. Any unexpended balances of funds so advanced shall be returned to the credit of the appropriation from which the funds have been advanced: *Provided*, That any advance made to any State under the provisions of this section and not repaid shall be deducted from any apportionment allocated to such State under the provisions of this Act for the year next succeeding the year in which such advance is made, and no agreement made in accordance with the provisions of section 2 of this Act shall be valid for any pro rata share of the cost of construction in excess of such apportionment less such advance.

SEC. 7. There shall be designated within the continental United States a National System of Interstate Highways not exceeding forty thousand miles in total

extent so located as to connect by routes, as direct as practicable, the principal metropolitan areas, cities, and industrial centers, to serve the national defense, and to connect at suitable border points with routes of continental importance in the Dominion of Canada and the Republic of Mexico. The routes of the National System of Interstate Highways shall be selected by joint action of the State highway departments of each State and the adjoining States, as provided by the Federal Highway Act of November 9, 1921, for the selection of the Federal-aid system. All highways or routes included in the National System of Interstate Highways as finally approved, if not already included in the Federal-aid highway system, shall be added to said system without regard to any mileage limitation.

SEC. 8. With the approval of the Federal Works Administrator, not to exceed 1½ per centum of the amount apportioned for any year to any State under the Federal Highway Act, as amended and supplemented, except sections 3 and 23 thereof, shall hereafter be used with or without State funds for surveys, plans, engineering, and economic investigations of projects for future construction in such State, on the Federal-aid highway system and extensions thereof within municipalities, on secondary or feeder roads, urban highways or grade-crossing eliminations, and for highway research necessary in connection therewith.

SEC. 9. For the purpose of carrying out the provisions of section 23 of the Federal Highway Act (42 Stat. 218), as amended and supplemented, there is hereby authorized to be appropriated (1) for forest highways the sum of $25,000,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years; and (2) for forest development roads and trails the sum of $12,500,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years: *Provided*, That the apportionment for forest highways in Alaska shall be for each year $1,500,000 and that such additional amount as otherwise would have been apportioned to Alaska for each of said years shall be apportioned among those States, including Puerto Rico, whose forest highway apportionment for such year otherwise would be less than 1 per centum of the entire apportionment for forest highways for that year.

SEC. 10. (a) For the construction, reconstruction, improvement, and maintenance of roads and trails, inclusive of necessary bridges, in national parks, monuments, and other areas administered by the National Park Service, including areas authorized to be established as national parks and monuments, and national park and monument approach roads authorized by the Act of January 31, 1931 (46 Stat. 1053), as amended, there is hereby authorized to be appropriated the sum of $12,750,000, to become available at the rate of $4,250,000 a year for each of the three successive post-war fiscal years.

(b) For the construction and maintenance of parkways, to give access to national parks and national monuments, or to become connecting sections of a national parkway plan, over lands to which title has been transferred to the United States by the States or by private individuals, there is hereby authorized to be appropriated the sum of $30,000,000, to become available at the rate of $10,000,000 a year for each of the three successive post-war fiscal years.

(c) For the construction, improvement, and maintenance of Indian reservation roads and bridges and roads and bridges to provide access to Indian reservations and Indian lands under the provisions of the Act approved May 26, 1928 (45 Stat. 750), there is hereby authorized to be appropriated the sum of $6,000,000 for the first post-war fiscal year and a like amount for each of the second and third post-war fiscal years: *Provided*, That the location, type, and design of all roads and bridges constructed shall be approved by the Public Roads Administration before any expenditures are made thereon, and all such construction shall be under the general supervision of the Public Roads Administration.

SEC. 11. Federal highway funds shall not be used for the reconstruction or relocation of any highway giving access to an airport (if such airport has been constructed or extended after the date of enactment of this Act), or for the reconstruction or relocation of any highway which has been or may be closed or the usefulness of which has been or may be impaired by the location or construction of any airport (if such airport has been constructed or extended after the date of enactment of this Act), unless, prior to such extension or construction, as the case may be, the State highway department and the Public Roads Administration have concurred with the officials in charge of the airport that the location of such airport or extension thereof and the consequent reconstruction or relocation of the highway are in the public interest.

SEC. 12. On any highway or street hereafter constructed with Federal aid in any State, the location, form, and character of informational, regulatory, and warning signs, curb and pavement or other markings, and traffic signals installed or placed by any public authority, or other agency, shall be subject to the approval of the State highway department with the concurrence of the Public Roads Administration; and the Commissioner of Public Roads is hereby directed to concur only in such installations as will promote the safe and efficient utilization of the highways.

SEC. 13. If any section, subsection, or other provision of this Act or the application thereof to any person or circumstance is held invalid, the remainder of this Act and the application of such section, subsection, or other provision to other persons or circumstances shall not be affected thereby.

SEC. 14. This Act may be cited as the "Federal-Aid Highway Act of 1944".

Approved December 20, 1944.

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